NUMERICAL MODELING OF ALONGSHORE SEDIMENT TRANSPORT AND SHORELINE CHANGE ALONG THE GALVESTON COAST

A Thesis

by

KHAIRIL IRFAN SITANGGANG

Submitted to the Office of Graduate Studies of
Texas A&M University
in partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE

December 2003

Major Subject: Ocean Engineering

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Approved as to style and content by:

Billy L. Edge (Co-Chair of Committee)	Thomas M. Ravens (Co-Chair of Committee)
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ABSTRACT

Numerical Modeling of Alongshore Sediment Transport

and Shoreline Change Along the Galveston Coast. (December 2003)

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Dr. Thomas M. Ravens

An alongshore sediment transport and shoreline change analysis on Galveston Island in the period of 1990-2001 is conducted in this study using the Generalized Model for Simulating Shoreline Change (GENESIS). The study is divided into three main parts: 1. Assessment of the numerical accuracy of GENESIS, 2. Assessment of the alongshore sediment transport and shoreline change on the Galveston coast in the period of 1990-2001, and 3. Assessment of several erosion control practices on the Galveston coast for the period of 2001-2011.

The first assessment shows that GENESIS has a numerical error which tends to be large for low energy wave (small breaking wave height) and large breaking wave angle. This numerical inaccuracy cannot be neglected and needs to be compensated for. This can be done, for instance, by adjusting the transport parameter K1.

In the second assessment, good agreement between the calculated and measured transport/shoreline is achieved, particularly on the West Beach. Comparison between the

potential alongshore sediment transport and sediment budget-inferred alongshore transport provides a systematic way of selecting the proper wave data set for the alongshore and shoreline change calculation.

The third assessment proves that beach nourishment is the best alternative to overcome/reduce the erosion problem on the Galveston coast. Constructing coastal structure (groins, offshore breakwater) on the West Beach does not resolve the problem of erosion, but instead shifts it further west.

This thesis is dedicated to my mother, father, family, and my beloved wife, Dine.

ACKNOWLEDGMENTS

Many people have contributed to this study. It would have been impossible to complete it on time without them. I would like to express my gratitude to Dr. Tom Ravens for his patience in assisting me during this research and for providing me with a research assistantship to support my study of Ocean Engineering at TAMU. I would also like to thank Dr. Billy Edge and Dr. Robert Stewart for their kindness in serving as co-chair and committee member on my thesis committee. I am very grateful for the encouragement and support of Dr. Vijay Panchang of Maritime System Engineering, TAMUG.

During this research, much help came from Randall Thomas in providing the bathymetry data. Thanks Randy for your help and warm friendship. Discussion of CGWAVE with Dongcheng Li and Dr. Zeki Demirbilek is really appreciated. I would like to thank Noah and Looke for their nice and warm friendship during our stay in Galveston. To Mas Pandoe, thanks for your support and useful discussions on a lot of topics related to this research.

Finally I would like to express my gratitude and respect to my parents and family, and special thanks to my beloved wife, *Dine*.

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CHAPTER I

INTRODUCTION

Background

Galveston Island is a barrier island located at (–94.83°, 29.25°) in the upper Texas Gulf of Mexico. The island is about 30 miles long and 3 miles wide in the middle. It borders on West Bay in the north, Gulf of Mexico in the south, Bolivar Roads in the east, and San Luis Pass in the west (Figure 1).

Construction of various coastal structures on the island started decades ago. In 1887, the U.S. Army Corps of Engineers began a project to build jetties from the eastern tip of Galveston Island and the western tip of Bolivar Peninsula out into the Gulf of Mexico. The jetties were completed by around 1907; the North Jetty extended about 5 kilometers and the South Jetty extended about 4 kilometers into the Gulf of Mexico (Figure 1). The purpose of constructing the North and South Jetty was to keep the Bolivar Roads navigable. Before the construction of the jetty, sandbars were developing in the harbor entrance, forcing some ships to have to wait for high tide before they could enter the harbor. Following the catastrophic hurricane in 1900, which took more than 6,000 lives, Galveston seawall was constructed. The construction began in 1902 and was completed in the ensuing years. The seawall stretches 10 miles westward from the South Jetty. This seawall protects the island from hurricanes. The remaining coastal structures built on the island constitute the groins in the groin field. The construction was initiated

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This thesis follows the style and format of *Journal of Coastal Research*.

in 1936 and completed in 1939. The present groin field consists of a total of 17 groins, which measure approximately 152 meters long (except for the westernmost unit off 59th street which is 91 meters long). They are spaced about 457 meters apart. It occupies a 4.5-kilometer shoreline segment. The groins have acted to reduce the erosion in the groin field (SONU *et al*., 1979).

The island has been very dynamic, particularly on the southern side of the island that is adjacent to the Gulf. With the exception of the East Beach, the rest of the island has been subject to erosion in the past decades with rates varying from –0.6 m/yr in the groin field to –3 m/yr on the unprotected West Beach. The East Beach, under the shadow of the South Jetty, has been accreting with rate about +0.6m/yr (GIBEAUT *et al*., 1998). To respond to this erosion threat, the local government nourished the beach in front of the groin field in 1995 (574,000 m^3) and in the spring of 1998, 1999, and 2000 (54,000 m^3).

Purpose of Study

The purpose of this research is to study the dynamics of the Galveston coast using a mathematical model. The study starts with the analysis of alongshore sediment transport using the historical data (shoreline and wave). This analysis helps justify which mathematical model is appropriate to implement on Galveston coast and what assumptions should be made. In this study, cross-shore sediment transport due to big storm/hurricane is not taken into account. The focus will be put on the wave-generated alongshore sediment transport.

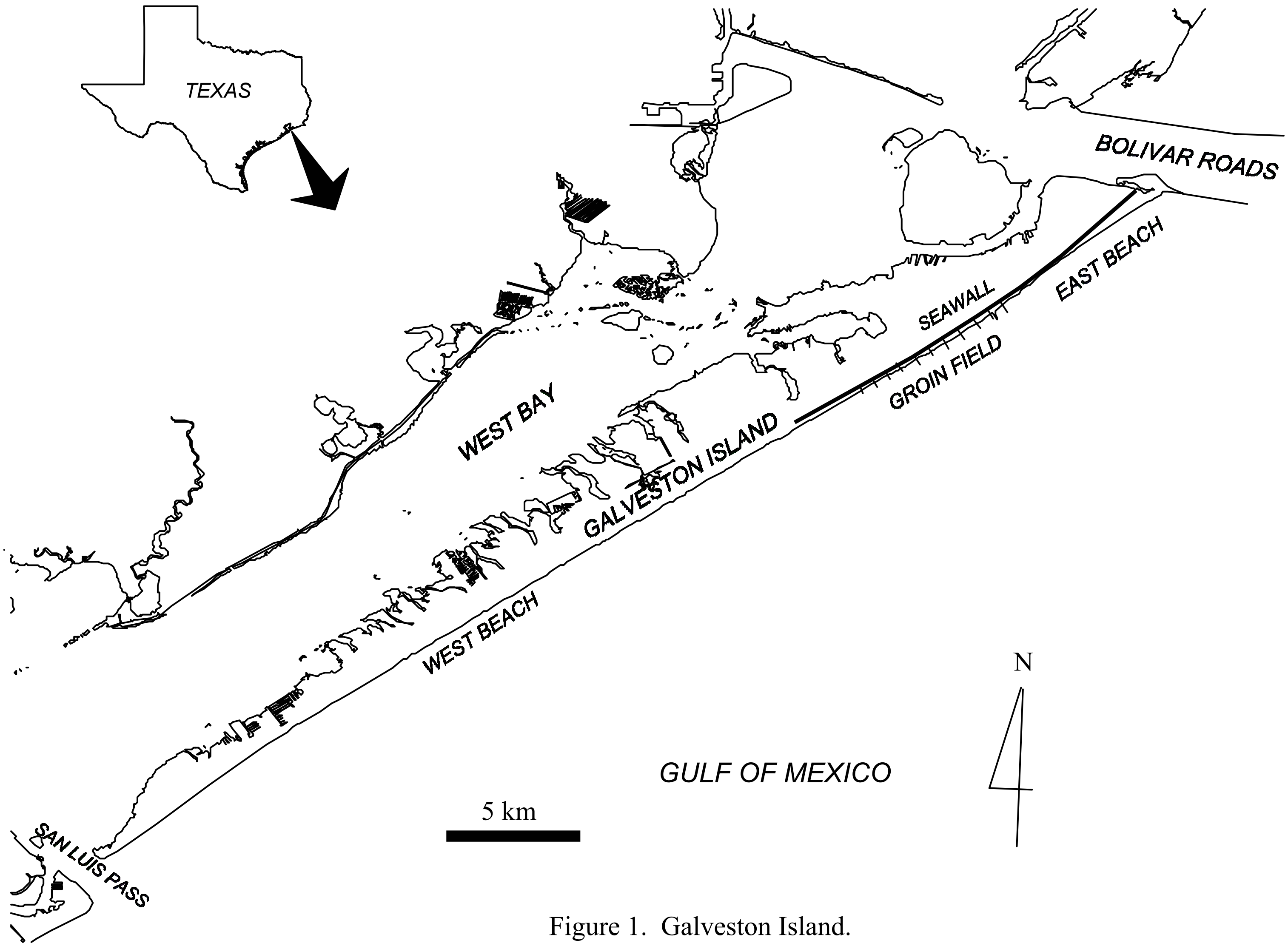

Figure 1. Galveston Island.

Objective of Study

The objectives of this study are:

1. To assess the applicability of GENESIS to study the alongshore sediment transport and shoreline change on Galveston Island.
2. To calculate the sediment transport along the Galveston coast in the period of 1990 to 2001 with primary focus on the alongshore sediment transport that is generated by the wave-induced momentum flux. The mathematical model of alongshore sediment transport that will be employed is the CERC formula. This calculation will be compared with the sediment transport calculation derived from sediment budget analysis which is based on the historical shoreline positions. This way, the validity of applying the CERC formula will be verified for this location.
3. To asses various erosion control options on Galveston Island.

Research Procedure

This study will be conducted through several steps as follows:

<u>Step 1</u>

Test GENESIS model against two idealized shoreline change problems which can be analytically solved. This test will find how well GENESIS predicts the shoreline change from a numerical point of view.

<u>Step 2</u>

Conduct sediment budget analysis for Galveston Island on the basis of the 1990 and 2001 shoreline data with the assumption that no cross-shore transport occurs during that period.

Step 3

Calculate the potential alongshore sediment transport (for the same period) using CERC formula and waves from the 1976-1995 hindcasting. GENESIS, which employs CERC sediment transport formula, will be used to perform this calculation.

Step 4

Screen the available wave data (step 3) by evaluating which wave years provide the "correct" sediment transport according to the sediment budget analysis (step 2).

Compare the sediment transport calculated in steps 2 and 3 to choose the most representative wave data to use for simulating the shoreline change over the period of 1990 to 2001.

Step 5

Calculate the alongshore sediment transport and shoreline change for the period of 1990-2001 using the "correct" wave conditions (step 4).

Previous Research

Much research has been conducted on the sediment transport and coastal hydrodynamics. In the very beginning step of this study, attempts are made to collect as much information from the previous related research as possible in order to better understand the problem at hand and to find a better strategy to solve them. The research that is most pertinent to this study is summarized below.

1. MORTON (1974) studied the shoreline changes on Galveston Island (Bolivar Roads to San Luis Pass) on the basis of the compilation of shoreline and vegetation line positions from topographic maps, aerial photographs, and coastal charts of various

dates. Long-term trends in shoreline change averaged over 135-year time period indicated variation of shoreline change rates along the Galveston coast. The East Beach (from the South Jetty to the seawall) experienced accretion; maximum net accretion was greater than 1,829 meters. The groin field was eroding with net erosion ranging from 9 to 267 meters and so was the stretch of beach from the end of the seawall to 6 kilometers to the west; its erosion rate was 0.4 meter/year to 3.2 meters/year. Minor net accretion was recorded along the next 7.6 kilometers of beach, with averaged net to be 23 meters (rates of change was less than 0.3 meter/year). The remaining 16-kilometer beach westward to San Luis Pass, experienced erosion about –0.3 to –0.5 meter/year. The major factors affecting shoreline changes along the Texas Coast, including Galveston Island, were a deficit in sediment supply and relative sea-level rise or compaction subsidence. Changes in the vegetation lines were primarily related to storms.

2. HALL (1976) studied the alongshore sediment transport along Bolivar Peninsula and Galveston Island. In calculating the alongshore sediment transport, he used the breaking wave condition to calculate the alongshore component of wave energy flux, which is related through an empirical curve to alongshore transport rate (CERC formula). This formula, with transport constant K = 0.81, was found to be good for the alongshore sediment transport calculation in his study area. He found that during 1975 the alongshore transport along Bolivar Peninsula and Galveston Island was toward the southwest at rates that varied from 116,000 m^3/yr at Sea Isle on Galveston Island to 41,000 m^3/yr at Gilchrist on Bolivar Peninsula. Net alongshore transport

was toward the southwest during months of fall, winter, and spring and generally toward the northeast only during three months of summer.

3. SONU, *et al*. (1979) performed a study on beach processes over the 161-kilometer shoreline between Sabine Pass and the Brazos River by means of literature review, review and reanalysis of existing data, analysis of recent shoreline changes based on a new set of data since 1939, and sediment budget analysis. They found that the shoreline change over 15-year period of 1960 to 1975 on the East Beach showed an accretion rate of +4 meters/year. For the same period of time, the beach on the groin field also showed accretion with rate of +2.6 meters/year. The West Beach, however, suffered from erosion, rated about –3.4 meters/year during that 15-year period.
4. GILBREATH (1995) studied the shoreline change and the alongshore sediment transport along the East Beach, groin field, and West Beach. Using GENESIS he calculated the shoreline changes on the three big compartments. He used the shorelines from the 1978/1982 measurement for studying the shoreline change in the East Beach, the 1977/1979 and 1976/1980 measurement for studying the shoreline changes in the groin field, and the West Beach respectively. He found that within 10 years, 454,000 m^3 of sand was deposited in the East Beach with the overall shoreline accretion of 0.84 meter. A much smaller deposition of 6,410-m^3 sand occurred in the groin field within the same period of time, which led to 0.06-meter shoreline advance over the entire groin field. The West Beach, on the other hand, experienced big erosion about 1,410,000 m^3 in ten years, which caused shoreline retreat of –0.50 meter/year.

5. LARSON *et al*. (1997) analytically solved several idealized shoreline change problems. In modeling the shoreline change they used the one-line theory and made some simplifications so that the governing equation of the problem was analytically solvable in terms of compact mathematical formulas. The problems they solved include: (1) shoreline change at detached breakwater, (2) shoreline change at semi-infinite seawall, (3) shoreline change at single groin with varying wave direction, (4) shoreline change in groin compartment with varying wave direction, (5) shoreline change at jetty, including bypassing, and (6) shoreline change at jetty, including diffraction.
6. GIBEAUT *et al*. (1998) calculated the shoreline change rates of the Gulf of Mexico on the basis of the past shoreline positions. A computer program called the Shoreline Shape and Projection Program (SSAP), developed by the Bureau of Economic Geology of The University of Texas at Austin, was used to calculate the change rate of segmented baseline that follows the mean position of the historical shorelines. They used the 1956, 1970, 1990, and 1996 shoreline to calculate the shoreline change rate on the East Beach, the 1956, 1965, and 1990 shoreline to calculate the shoreline change rate in front of the Galveston seawall, and the 1956, 1965, 1974, 1990, and 1996 shoreline to calculate the shoreline change rate on the West Beach. This study revealed that the rates of shoreline change on the East Beach, groin field, and West Beach were found to be +0.6 meter/year (accretion), –1.3 meters/year (erosion), and –2.8 meters/year (erosion) respectively.

7. WANG *et al*. (1998) conducted measurement of alongshore sediment transport using streamer traps on 29 locations along the southeast coast of the United States and the Gulf of Florida. They also performed a concurrent measurement using traps and short-term impoundment at Indian Rocks Beach, west-central Florida. Data on beach profiles, breaking wave conditions, and sediment properties were also taken together with the transport rate. Comparison of their measurement with the published empirical formula (CERC formula), which was calibrated on the (high-wave energy) Pacific coast, indicated that the formula overestimated the sediment transport in the low-wave energy coasts. They found a linear relationship between the alongshore transport and wave energy flux factor similar to the commonly used CERC formula. However, lower empirical constant of 0.08 instead of the nominal value 0.78 (recommended by Shore Protection Manual) was more suitably used to calculate the transport in lower-wave energy coast.
8. WANG and KRAUS, N.C. (1999) measured the alongshore sediment transport rate in the surf zone of Indian Rocks Beach (west central Florida) using short-term impoundment at temporarily installed groin. He found that the magnitude of the transports were considerably lower than predictions of CERC formula. He also arrived at a conclusion that the transport parameter K was not a constant and that other factors such as breaker type, turbulence intensity, and threshold for sediment transport might constitute to the transport mechanism.
9. THIELER *et al*. (2000) criticized the use of several mathematical models such as CERC formula and GENESIS in the circle of coastal engineering practitioners for

the purpose of coastal engineering planning and design. To sum up, he criticized the use of several assumptions such as depth of closure, beach equilibrium profile, uniform distribution of Q along the surf zone, no cross-shore transport, temporal and spatial grain size not considered, storm effect not included, wave shape/breaker type not considered, etc. He emphasized that coastal engineering practitioners should be aware of all the assumptions made in the models and the exclusion of some variables in the model when they are planning and designing coastal works.

10. RAVENS and SITANGGANG (2002) developed the shoreline change model (GENESIS) of the Galveston shoreline with a particular focus on the Galveston nourishment project. They calibrated the model on the basis of the shoreline measurement in the period of 1965 to 1990. They also assessed the effects of putting T-groins, offshore breakwaters in the groin field, and doubling the volume of the nourishment. They found that the groin field area was very sensitive to the shoreline control practices such as noted above.

CHAPTER II

SHORELINE CHANGE MODEL (GENESIS)

GENESIS is an acronym that stands for Generalized Model for Simulating Shoreline Change (HANSON and KRAUS, 1989). This model is designed to simulate the long-term shoreline change at coastal engineering project. The alongshore extent of a typical modeled reach can be in the range of 1 to 100 kilometers and the time frame of a simulation can be in the range of 1 to 100 months. Coastal structures such as groins, detached breakwaters, seawalls, and jetties and beach nourishments can be represented in the model. GENESIS contains what is believed to be a reasonable balance between present capabilities to efficiently and accurately calculate coastal sediment processes from engineering data and the limitations in both the data and knowledge of sediment transport and beach change. GENESIS simulates shoreline change produced by spatial and temporal differences in alongshore sand transport.

Standard Assumptions

Several assumptions were made in developing the shoreline change model (GENESIS). These assumptions simplify the complex shoreline change problem and make it solvable in terms of a relatively simple mathematical model. In general, the standard assumptions made in the shoreline change model are:

1. The beach profile shape is constant.

2. The shoreward (berm height D_B) and seaward limit (depth of closure D_C) of the beach profile are constant.
3. The alongshore sand-transport is produced by the energy of the breaking wave.
4. The detailed structure of the near-shore circulation is ignored.
5. There is a long-term trend in shoreline evolution.
6. Alongshore transport is uniform across the surf zone.

The first assumption indicates that the beach profile moves (seaward and shoreward) parallel to itself without changing shape in the course of eroding and accreting. Thus, one contour line is sufficient to describe the change in the beach plan shape and volume as the beach erodes and accretes. The model with this assumption is sometimes termed as "one line model".

The second assumption specifies the region of active alongshore transport, which is bounded by two constant limiting elevations. Outside this region the alongshore transport (geometric change) is insignificant. The shoreward limit is located at the top of the active berm and the seaward limit is located at the so-called depth of closure.

The third assumption attempts to express the alongshore transport on the beach in terms of the breaking wave condition[1]. Since the breaking wave condition is used, detailed structure of near-shore circulation is ignored (assumption 4).

The fifth assumption indicates that the model predicts the steady long-term shoreline change only; such short-term change produced by storms, seasonal changes in

[1] Another approach is to use the shear stress approach, where the sediment entrainment is produced by the shear stress alone.

waves, tidal fluctuations, and other cyclical and random events (short-term changes) are not considered.

The last assumption indicates no cross-shore variation of alongshore transport across the surf zone.

With the above assumptions made in the shoreline change model (GENESIS), one must be cautious in using it as a tool to predict the shoreline change and careful interpretation of the calculation results must be made.

Governing Equation of Shoreline Change

The governing equation of the shoreline change model is derived on the basis of conservation of sand mass. This equation is mathematically expressed as follows:

$$\frac{\partial y}{\partial t} + \frac{1}{D_B + D_C}\left(\frac{\partial Q}{\partial x} - q\right) = 0\,, \qquad (2\text{-}1)$$

where

y	=	shoreline position from the x-axis
t	=	time
x	=	alongshore distance
D_B	=	berm height relative to water level
D_C	=	depth of closure relative to water level
Q	=	alongshore transport rate
q	=	line source or sink of sand.

To solve Equation (2-1), an initial condition, i.e. the shoreline position along the simulation reach, two boundary conditions on both sides of the simulation reach, and the values of Q, q, D_B, and D_C must be provided.

Boundary conditions at both ends of the simulation reach may have various forms. The boundary conditions reflect the nature of the beach at the boundaries. Three boundary conditions are identified in GENESIS:

1. Pinned boundary condition. This boundary condition holds the shoreline at its initial position at the boundary along the simulation period. It is used to specify a beach without significant change in position during a given period of time.
2. Gated boundary condition. This boundary condition is used to model the blockade of alongshore transport by a jetty. It allows the control of the amount of sand transported across the jetty.
3. Moving boundary condition. This boundary condition is used on a beach with known shoreline change during a given period of time.

Alongshore Sediment Transport

The alongshore sediment transport rate which enters equation (2-1) must be calculated during the shoreline change calculation. The alongshore transport rate which is employed in GENESIS is mathematically expressed as follows:

$$Q = (H^2 C_g)_b \left\{ a_1 \sin 2\theta_{bs} - a_2 \cos\theta_{bs} \frac{\partial H}{\partial x} \right\}_b , \qquad (2\text{-}2)$$

where

H = wave height

C_g = wave group velocity given by linear wave theory

b = subscript denoting breaking wave condition

θ_{bs} = angle of breaking wave to the local shoreline.

The non-dimensional parameters a_1 and a_2 are given by:

$$a_1 = \frac{K_1}{16(\rho_s/\rho - 1)(1-p)(1.416)^{5/2}}$$

and

$$a_2 = \frac{K_2}{8(\rho_s/\rho - 1)(1-p)\tan\beta(1.416)^{7/2}}, \tag{2-3}$$

where

K_1, K_2 = empirical coefficients, treated as calibration parameters

ρ_s = density of sand (taken to be 2.65×10^3 kg/m^3 for quartz sand)

ρ = density of water (1.03×10^3 kg/m^3 for seawater)

p = porosity of sand on the bed (taken to be 0.4)

$\tan\beta$ = average bottom slope from the shoreline to the depth of active alongshore sand transport.

The factor 1.416 is used for the conversion of significant wave height (used by GENESIS) into root-mean-squared (rms) wave height (used by CERC formula).

The first term of equation (2-2) corresponds to CERC alongshore sediment transport formula and the second term, which is not part of CERC formula, is used to describe the effect of the alongshore gradient of breaking wave height on the alongshore transport.

The SHORE PROTECTION MANUAL (1984) suggested a value of K_1 = 0.78 and HANSON and KRAUS (1989) suggested a design value of K_1 typically lies within the range of 0.58 to 0.77. WANG *et al*. (1998) found K_1 = 0.08 (considerably different from both values above) in the lower wave energy coast in Florida. This indicates that K_1 is a wave-energy-dependent parameter rather than a constant as suggested by the first two references.

Equilibrium Profile

In calculating the alongshore sediment transport using equation (2-2) and (2-3), the breaking wave height and average bottom slope are required. GENESIS does not employ the actual bathymetry for the purpose of this calculation; instead it uses the so-called equilibrium profile. This equilibrium profile is mathematically written as follows:

$$D = Ay^{2/3}, \tag{2-4}$$

where D is the water depth and A (MOORE, 1982) is an empirical parameter that depends on the grain size and calculated as follows:

$$\begin{aligned} &A = 0.41(d_{50})^{0.94}, \ d_{50} < 0.4 \\ &A = 0.23(d_{50})^{0.32}, \ 0.4 \le d_{50} < 10.0 \\ &A = 0.23(d_{50})^{0.28}, \ 10.0 \le d_{50} < 40.0 \\ &A = 0.46(d_{50})^{0.11}, \ 40.0 < d_{50}, \end{aligned} \tag{2-5}$$

where d_{50} is the median grain size in mm and A in $m^{1/3}$.

The average bottom profile slope is determined by averaging the bottom slope of (2-4) from the shoreline up to the seaward limit of the littoral zone and found to be:

$$\tan\beta = \left(\frac{A^3}{D_{LTo}}\right), \tag{2-6}$$

where D_{Lto} is the depth at the seaward limit of the littoral zone.

Wave Transformation

The breaking wave condition is determined in GENESIS by transforming the wave from offshore reference point to breaking wave position. This transformation uses the linear wave theory (DEAN and DALRYMPLE, 1987).

Two procedures of calculating breaking wave condition are identified. In the first procedure, GENESIS transforms the wave from the offshore reference point to breaking wave location using the internal wave transformation. This transformation assumes a shore-parallel bathymetry. Wave transformation in a beach with nearly shore-parallel bottom profile may be modeled to a certain degree of accuracy using this procedure.

In the second procedure, GENESIS uses a separate external wave transformation model to do the wave transformation from offshore to near-shore reference line. This model uses the actual bathymetry. GENESIS does the rest of the job to transform the wave from near-shore reference (given by the external wave model) to breaking wave point, assuming shore-parallel bathymetry. This procedure is suitably used for a beach with significant bathymetry variation in alongshore direction.

STWAVE Wave Transformation Model

As mentioned previously, the external wave transformation model is used in the second procedure of calculating wave transformation. In this study, the Steady-State

Spectral Wave Model (STWAVE) is used to transform the wave from offshore to near-shore reference point.

STWAVE is a steady-state, phase-averaged spectral wave model based on the wave action balance equation. This model is capable of simulating depth-induced wave refraction and shoaling, current-induced refraction and shoaling, depth- and steepness induced wave breaking, simple diffraction, wave growth due to wind input, and wave-wave interaction and white capping that redistribute and dissipate energy in a growing wave field. More detail description of this wave model can be found in User's Manual for STWAVE Version 2.0 (SMITH *et al*., 1999).

CHAPTER III

MODEL TEST

Analytical Solution of Shoreline Change Problem

Under idealized situations, the governing equation of the shoreline change problem (2-1) may be analytically solved. LARSON *et al*. (1997) analytically solved several shoreline change problems with structures (i.e. groin, seawall, breakwater) present in the domain. In addition to their merits in providing a qualitative and quantitative insight of the future beach plan shape, the analytical solutions can also be used for the purpose of testing the accuracy of a numerical model such as GENESIS.

Here, two analytical solutions of shoreline change problem from the above source are used to test GENESIS performance. These problems and the corresponding analytical solutions are briefly presented as follows (interested readers are referred to LARSON *et al*., 1997).

1. Shoreline change at single groin with varying wave direction

In this problem, a single groin is present in the domain. It is impermeable and extends long enough to the sea so as not to let the sand pass its tip from one side to another. Hence, $Q = 0$ at the groin is acting as the boundary condition. Note that the groin is assumed not to diffract the wave propagating from offshore to breaking point. The breaking wave has a constant height and varies sinusoidally in direction in such a way that it generates a unidirectional alongshore transport. The breaking wave direction is written as:

$$\alpha_o(t) = \alpha_{ao}(1 + \sin \omega t), \qquad (3\text{-}1)$$

where

α_o = breaking wave angle

α_{ao} = amplitude of breaking wave angle

ω = angular frequency of the wave direction.

This idealized situation is depicted in Figure 2.

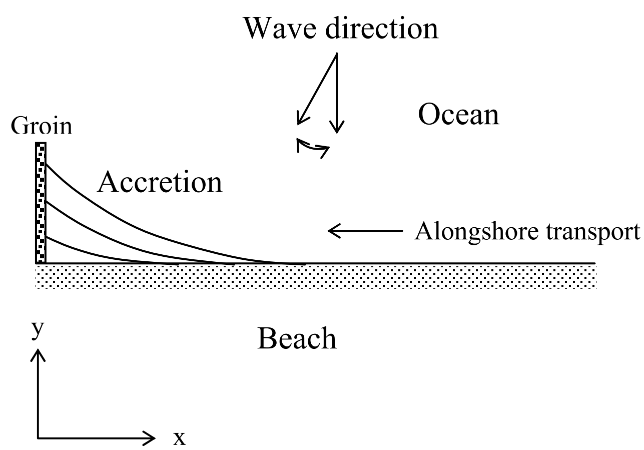

Figure 2. Domain and wave setup of shoreline change problem 1.

Assuming that the breaking wave crest makes a small angle to the shoreline and the shoreline slope relative to chosen coordinate (x-axis) is also small, the analytical solution to this problem can be written as follows:

$$y(x,t)=\alpha_{ao}\left[2\sqrt{\varepsilon t}\,\mathrm{ierfc}\left(\frac{x}{2\sqrt{\varepsilon t}}\right)+\frac{e^{-x\sqrt{\omega/(2\varepsilon)}}}{\sqrt{\frac{\omega}{\varepsilon}}}\sin\left(\omega t-x\sqrt{\frac{\omega}{2\varepsilon}}-\frac{\pi}{4}\right)+\frac{1}{\pi}\int_0^\infty\frac{\omega\cos\left(x\sqrt{\frac{\rho}{\varepsilon}}\right)e^{-\rho t}}{\sqrt{\frac{\rho}{\varepsilon}}(\rho^2+\omega^2)}d\rho\right], \tag{3-2}$$

where

$$\varepsilon = \frac{2Q_o}{D} \tag{3-3}$$

$$D = D_B + D_C \tag{3-4}$$

Q_o = amplitude of alongshore transport (amplitude of the sine function in CERC formula)

ρ = dummy variable of integration.

2. Shoreline change in groin compartment with varying wave direction

The second problem is dealing with the change of shoreline position between two impermeable and non-diffracting groins. These groins are long enough to completely blockade sediment transport. The breaking wave height is constant and the direction varies sinusoidally so as to generate alongshore transport in two opposite directions with zero net and is written as follows:

$$\alpha_o(t)=\alpha_{ao}\sin\omega t\,. \tag{3-5}$$

This idealized situation is depicted in Figure 3.

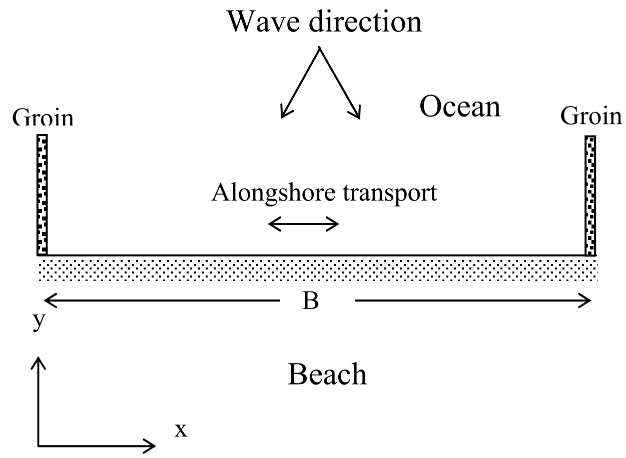

Figure 3. Domain and wave setup of shoreline change problem 2.

As with problem 1, the assumptions of small breaking wave angle to the shoreline and small slope of shoreline with respect to the chosen coordinate (x-axis) are also made in this problem. The analytical solution of this problem is mathematically written as follows:

$$y(x,t)=\frac{\alpha_{ao}\sqrt{\frac{\varepsilon}{\omega}}}{2\left[\cosh\zeta+\cos\zeta\right]}\left[e^{\zeta x/B}\sin\left(\omega t-\frac{\pi}{4}+\zeta\left(\frac{x}{B}-1\right)\right)+\right.$$
$$e^{\zeta[x/B-1]}\sin\left(\omega t-\frac{\pi}{4}+\zeta\frac{x}{B}\right)-e^{-\zeta[x/B-1]}\sin\left(\omega t-\frac{\pi}{4}-\zeta\frac{x}{B}\right)- \tag{3-6}$$
$$\left.e^{-\zeta x/B}\sin\left(\omega t-\frac{\pi}{4}-\zeta\left(\frac{x}{B}-1\right)\right)\right],$$

where

B = distance between the two groins

$$\zeta = \sqrt{\frac{\omega B^2}{2\varepsilon}}\,. \tag{3-7}$$

Comparison of the Numerical and Analytical Solution

In order to compare the solution given by the numerical and analytical method, similarities in all parameters involved in the calculation must be satisfied. For example, the wave conditions used in both solutions must be the same, i.e. breaking wave height, direction, and period, must have the same values in GENESIS and the analytical solution. To satisfy these criteria, in the following comparison the wave height and the depth at wave location are so chosen that the wave starts breaking, i.e.

$$\frac{H}{d} = 0.78, \tag{3-8}$$

Since GENESIS is provided with breaking wave condition, it does not need to carry out wave transformation to find breaking wave condition. Hence, similar wave height and direction can easily be setup for use in both GENESIS and the analytical solution.

Comparison 1

In this comparison, GENESIS is tested against the analytical solution (3-2). The error (as a percent) was found to be related to a single dimensionless number, Π[2]:

$$\Pi = \varepsilon\, t \,/\, \Delta x^2, \tag{3-9}$$

where Δx is the spatial (alongshore) discretization. The error was not found to vary significantly with Δt over the typical range of Δt used in GENESIS (1 to 6 hours).

The error analysis results in some important findings as follows:

1. The error is large for small Π and is small for large Π (see Figure 4).

---

[2] Π is determined based on the fact that the numerical error depends on ε, t, and Δx^2; combining the three parameters as shown in (3-9) gives a dimensionless parameter, Π, upon which the error depends. Using this parameter (instead of ε, t, and Δx^2) makes the error analysis easier.

2. The error has an asymptotic behavior for large Π (or large t) with error approaches ½α_{a0} (in degree), as may be observed in Figure 5.
3. The error is smaller for location closer to the groin and is larger for location farther from the groin (see Figure 6). The absolute error, however, is greatest near the groin.

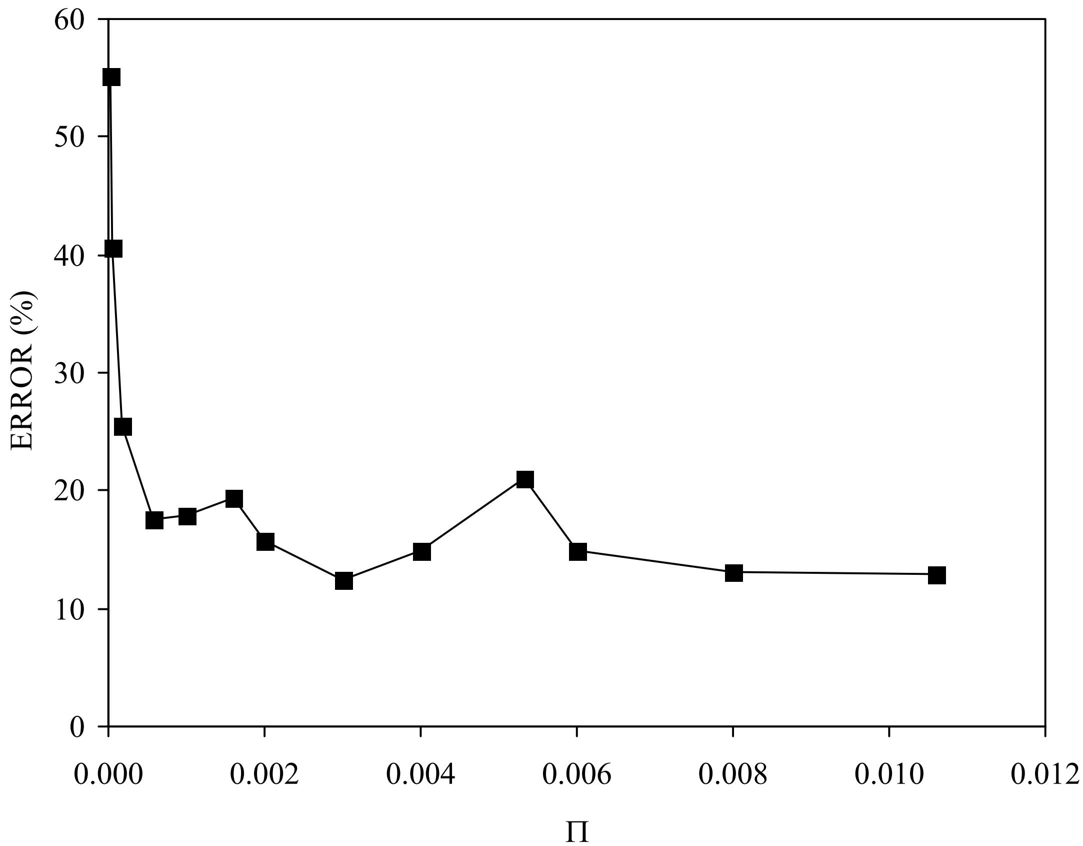

Figure 4. Error for various values of Π at x = 300 meters.

When the wave propagates from offshore to the beach, the direction changes so that at breaking the direction is usually near parallel to the beach, i.e. wave angle is small at breaking. Hence, as indicated by Figure 5, the numerical error will be small. If, for some reason (such as bathymetric configuration) the breaking wave is oblique to the beach, the numerical error introduced by GENESIS is not small, hence cannot be neglected. For low wave energy (low Π) coast, the numerical error is large and cannot be neglected either. The spatial grid size also influences the numerical accuracy, bigger grid size introduces more error than smaller grid size. To compensate for such error, K_1 might be appropriately justified.

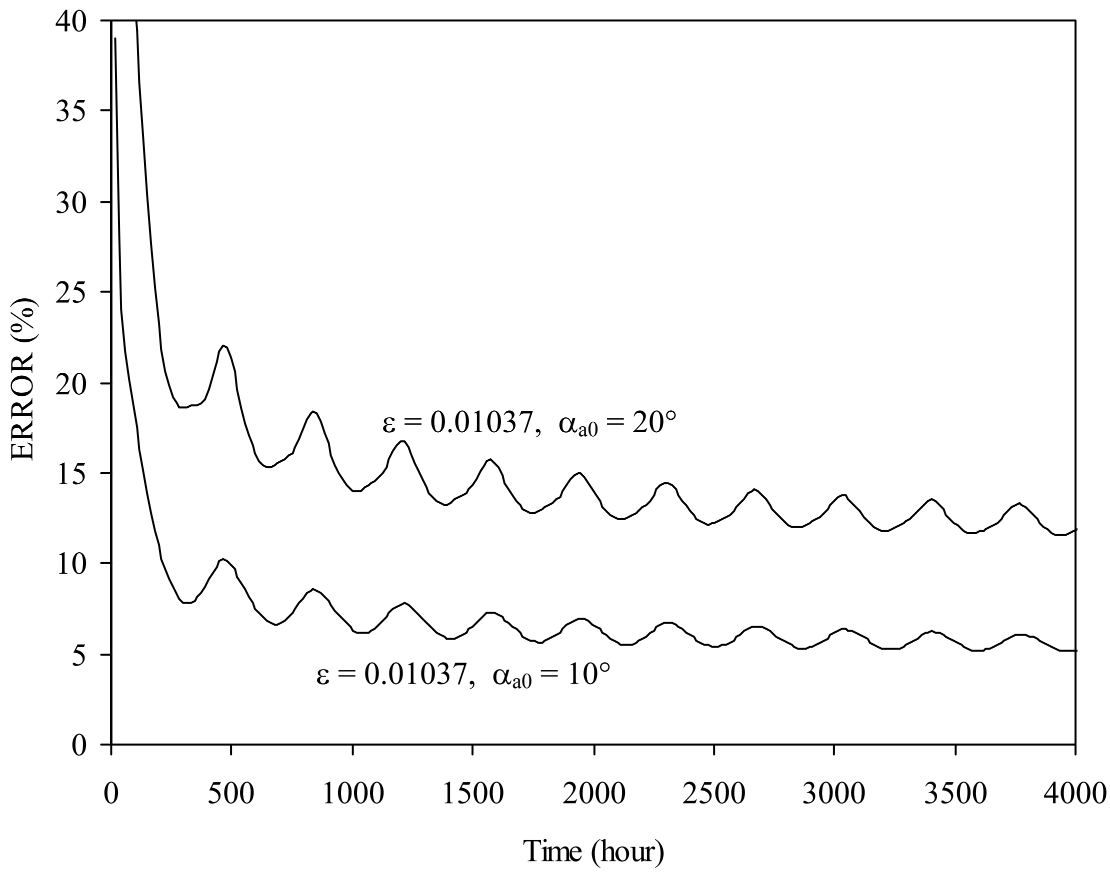

Figure 5. Error variation with time at x = 300 meters.

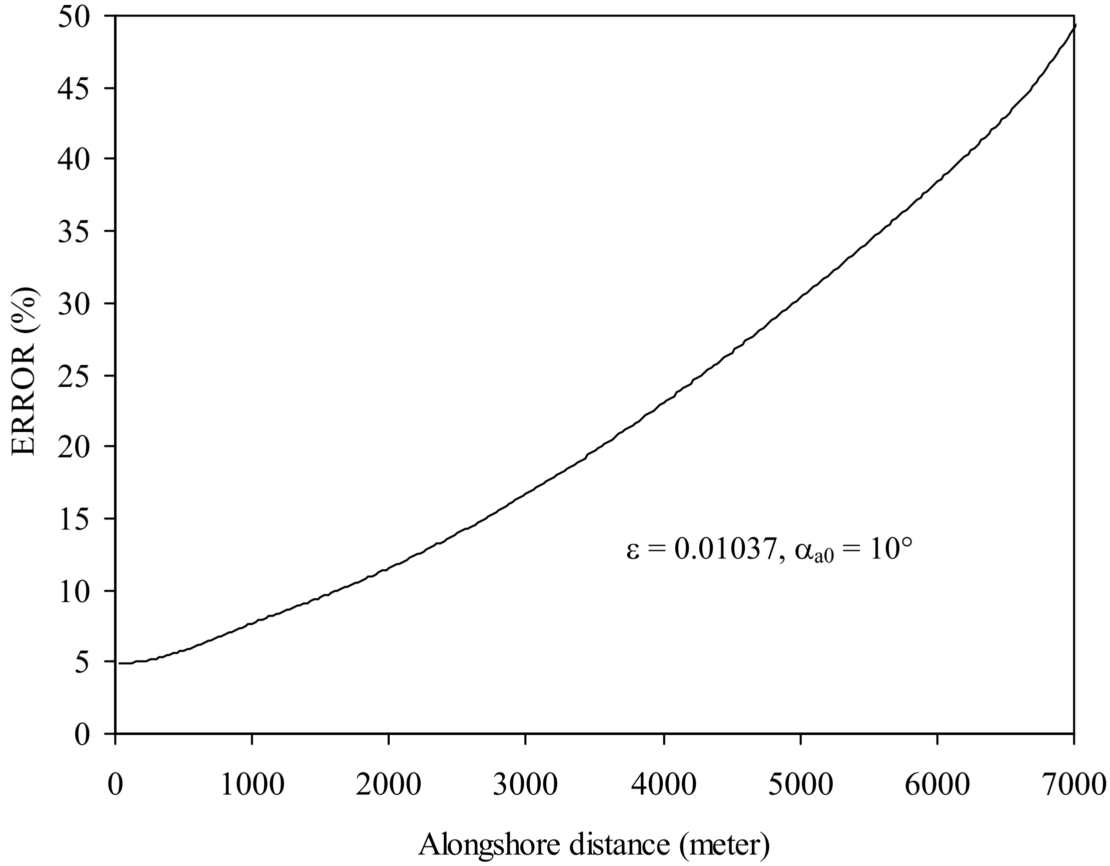

Figure 6. Alongshore variation of error at t = 10 years.

<u>Comparison 2</u>

This scenario tests GENESIS against the second analytical solution. The scenario is setup so that in the middle the beach is dynamic. If the wave energy is too small while the distance between the two groin is too large, the shoreline in the middle will be (near) static. This particular case is identical (except for the wave setup) to the first analytical problem.

In order to have a dynamic shoreline in all locations, wave height H = 0.6 m, wave period T = 4 s, wave angle amplitude α_{a0} = 30°, wave direction period Tdir = 360 days, the distance between the two groins, B = 1,200 m, and K_1 = 0.5 are used in this comparison. The spatial and time discretization are Δx = 25 m and Δt = 3 hours respectively. The initial condition for GENESIS calculation is the shoreline position calculated by the analytical solution at t = 0.

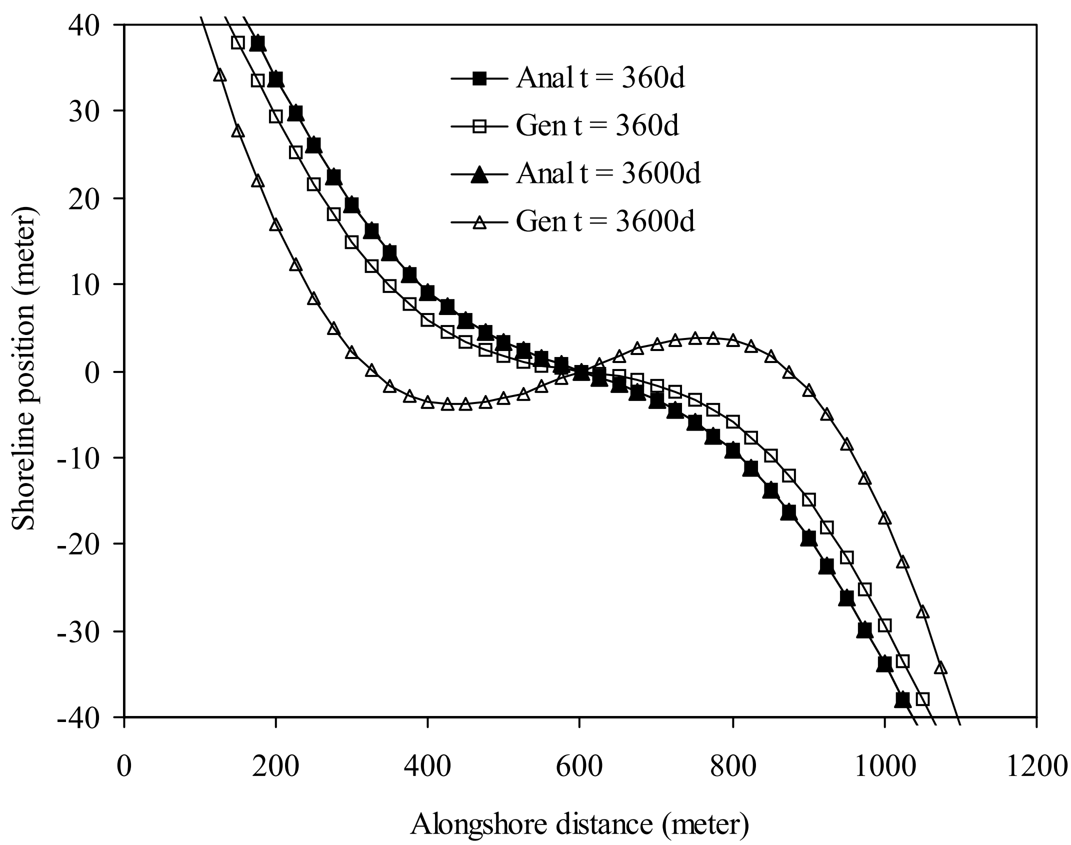

Figure 7. Analytically- and GENESIS- computed shoreline between two groins.

The shoreline position calculated by GENESIS and the analytical solution for this setup is depicted in Figure 7. As may be seen, the GENESIS-calculated shoreline at t = 360 days is in fairly good agreement with the analytically-computed shoreline, with a maximum absolute error of 4.6 meters. For longer simulation time the agreement is poor.

For example at t = 3,600 days, the maximum absolute error is 17.8 meters. The fact that GENESIS has problem with the numerical accuracy for a longer simulation time (in fact GENESIS is employed to study long term shoreline change) and particularly with the domain similar to this problem should be cautiously considered when making a long term shoreline change analysis using GENESIS.

CHAPTER IV

ALONGSHORE SEDIMENT TRANSPORT AND SHORELINE CHANGE CALCULATION

Wave Data

The wave records used in this study are obtained from the Wave Information Study (WIS) Station-1079, located in the Gulf of Mexico at (95°W, 29°N) and at depth of 15 meters. This station contains a total of 58,440 hindasted wave records, where the first record dates January 1, 1976 and the last record dates December 31, 1995.

Statistical analysis shows that most of the waves come from SE and SSE direction (see Figures 8, 9, 10, 11, 12, and Table 1). The analysis also reveals that in all years the wave distribution on the ENE-SE sector predominates over the wave distribution on the other SSE-SW sector. Note that both sectors are separated by shore-normal line (see Figure 8). Therefore, all waves in each individual year in general will generate westerly alongshore sand-transport along the Galveston coast. This agrees with previous observations (HALL, 1976).

Since the study area possesses several coastal structures that influence the alongshore sediment transport and shoreline change, such a simple statistical analysis is insufficient with respect to choosing the most representative wave data for simulating the dynamic of the coast. Hence, further analyses that take into account all structures in the study area must be carried out. These include the sediment budget and potential sediment transport analysis.

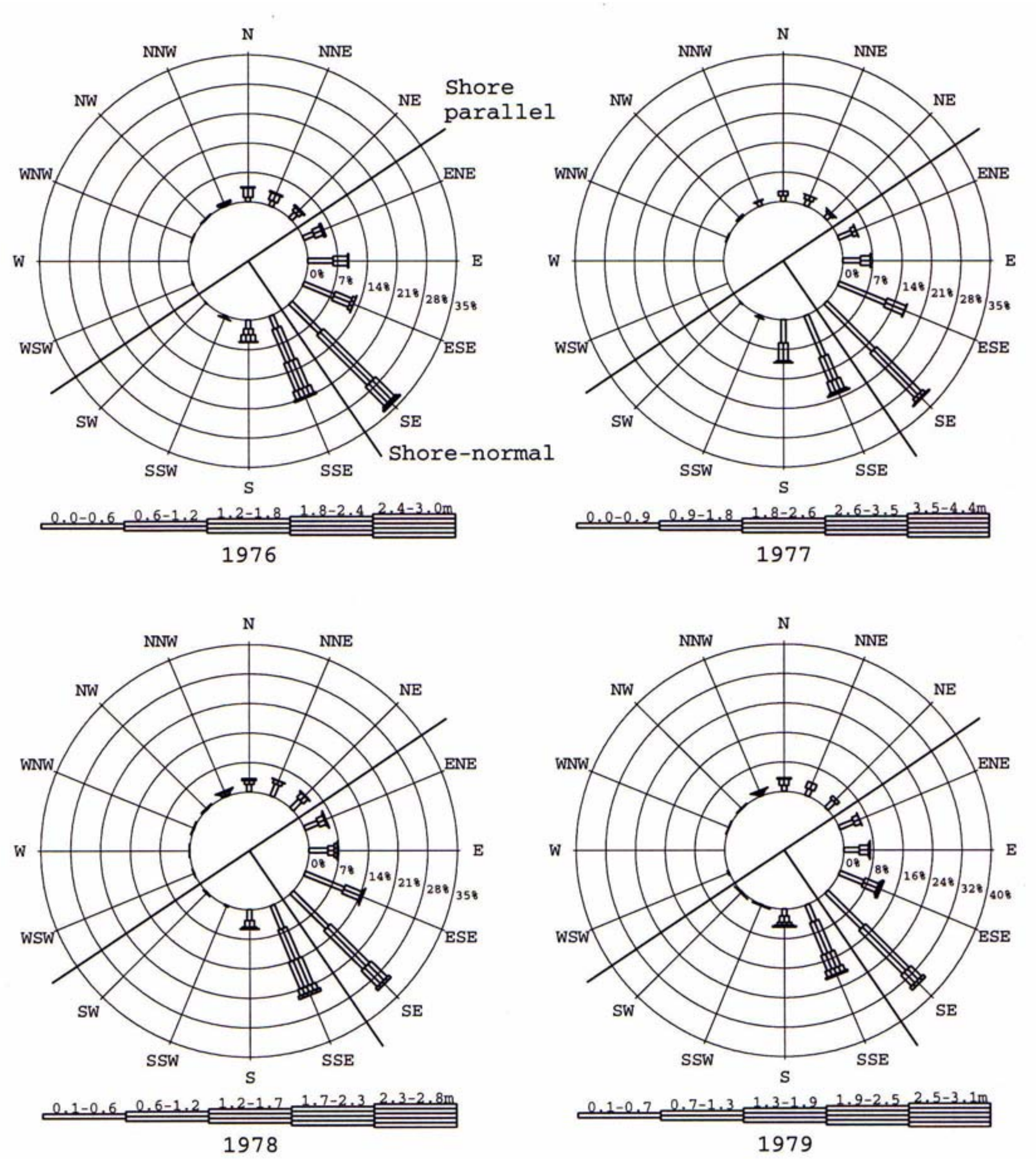

Figure 8. Wave roses of 1976, 1977, 1978, and 1979 waves in Station-1079.

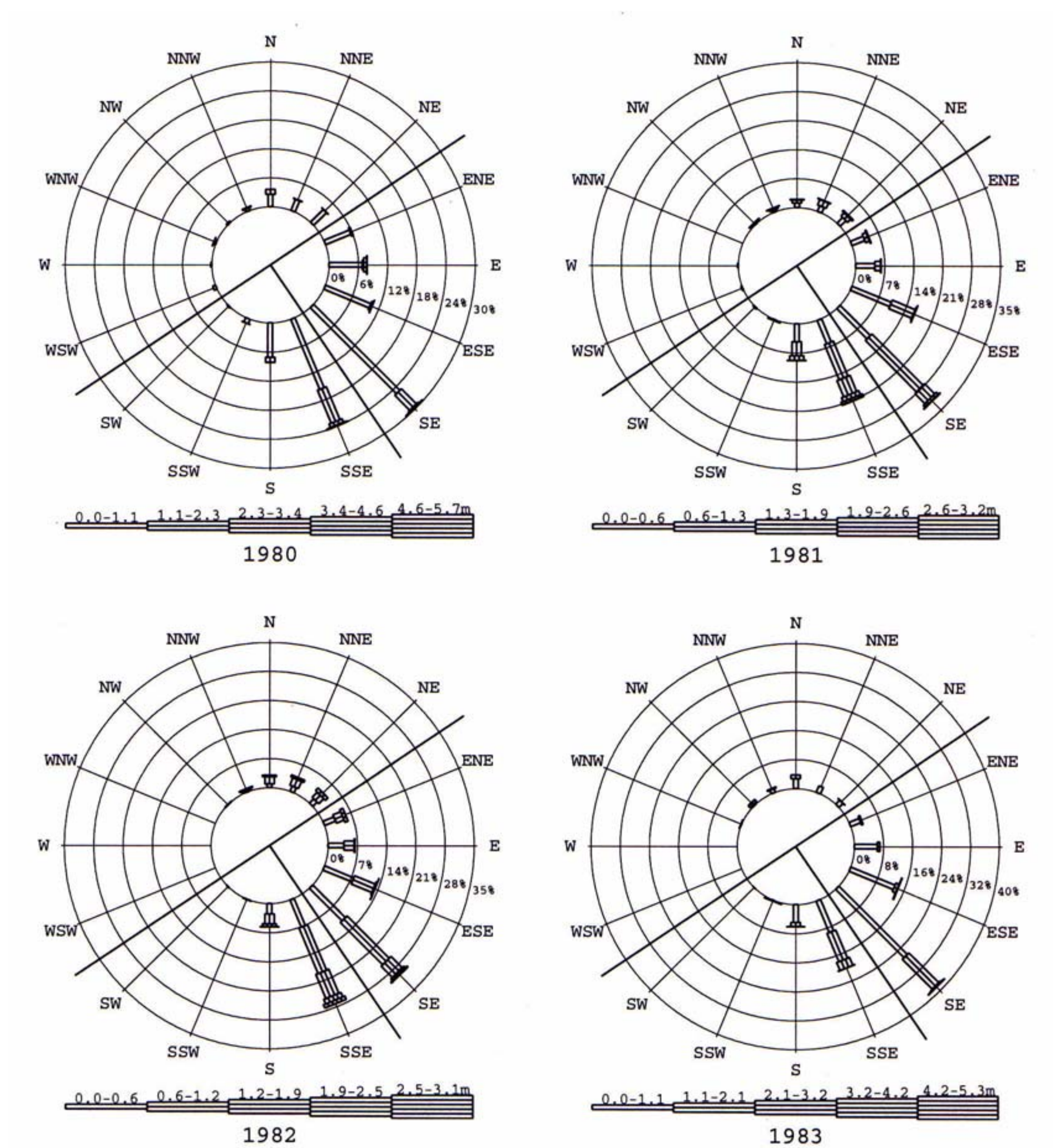

Figure 9. Wave roses of 1980, 1981, 1982, and 1983 waves in Station-1079.

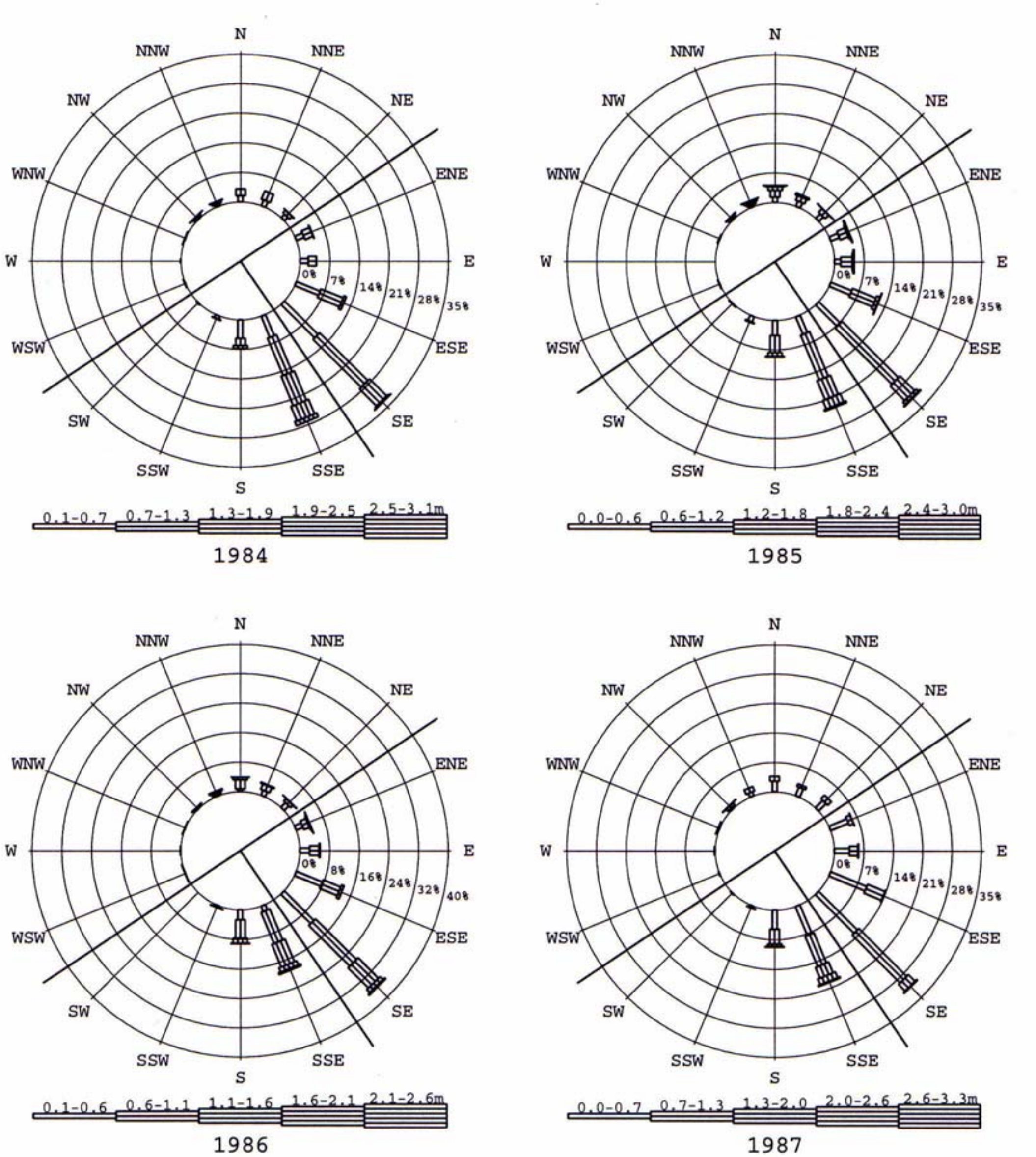

Figure 10. Wave roses of 1984, 1985, 1986, and 1987 waves in Station-1079.

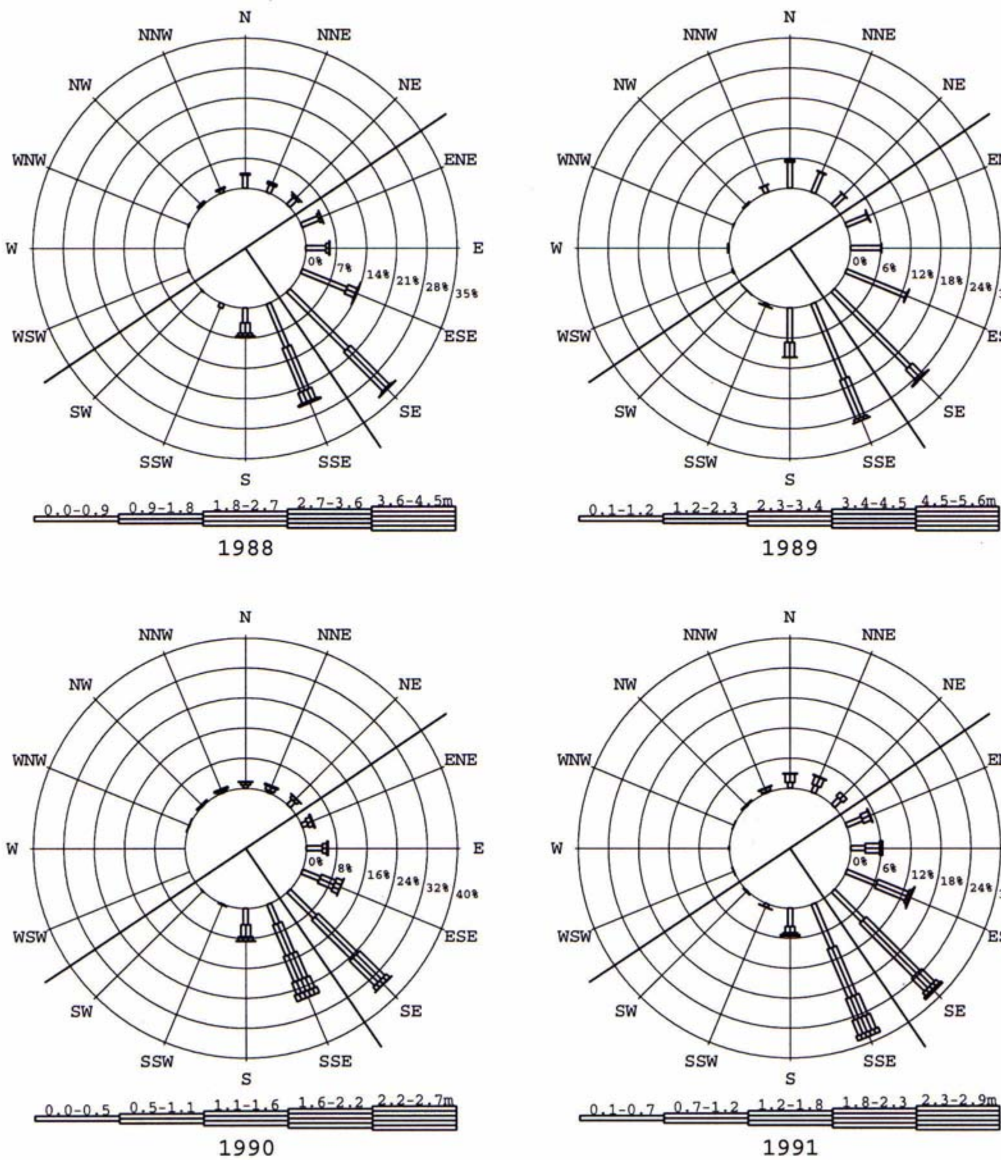

Figure 11. Wave roses of 1988, 1989, 1990, and 1991 waves in Station-1079.

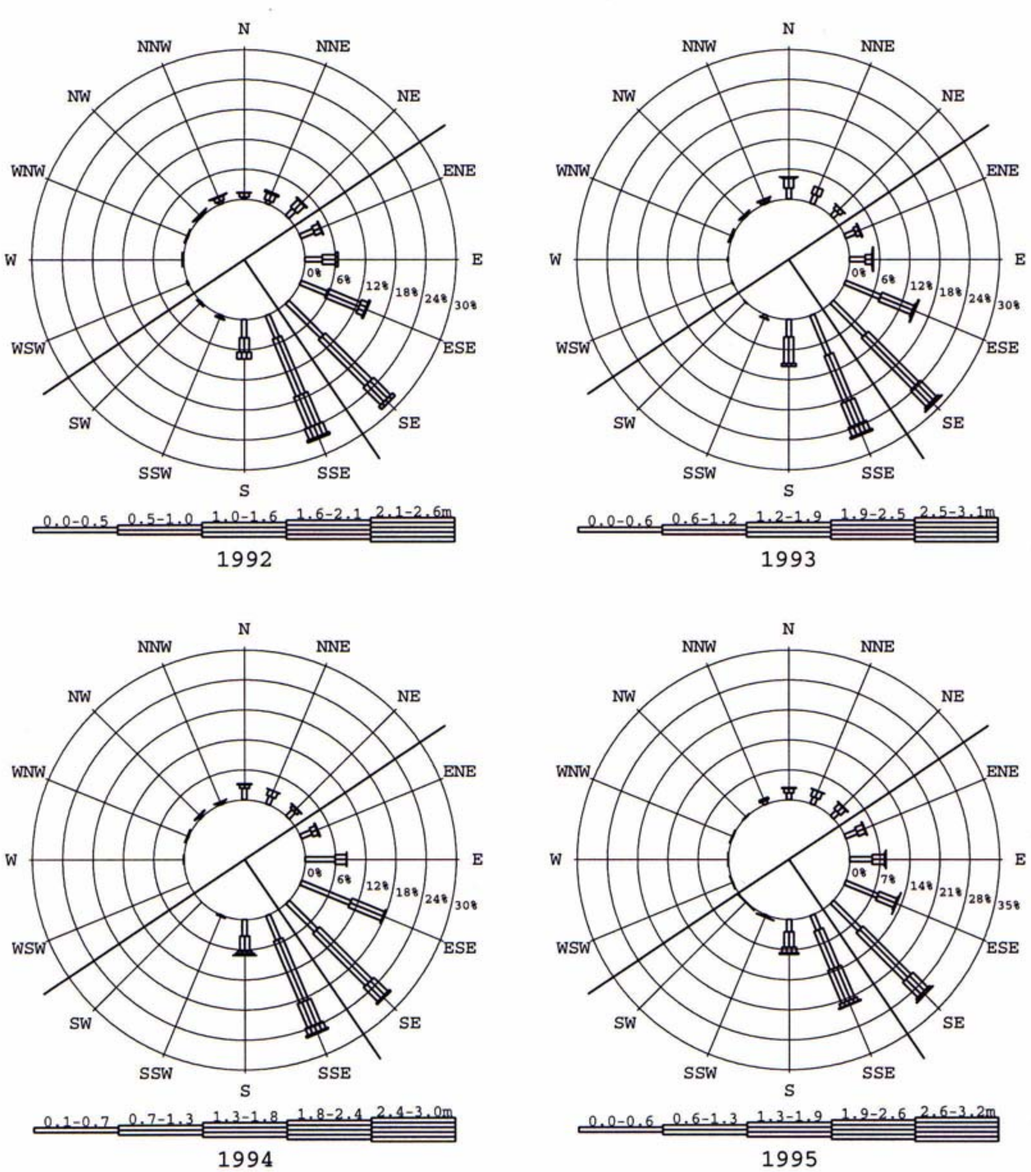

Figure 12. Wave roses of 1992, 1993, 1994, and 1995 waves in Station-1079.

Table 1. Wave distribution in WIS Station-1079 (all numbers are in %).

YEAR	ENE	E	ESE	SE	**TOTAL**	SSE	S	SSW	SW	**TOTAL**
1976	5.2	9.6	12.8	33.8	**61.4**	21.1	5.3	0.7	0.1	**27.2**
1977	4.5	6.6	17.0	32.1	**60.2**	20.0	10.2	0.7	0.1	**31.0**
1978	5.4	6.8	14.7	30.8	**57.7**	23.1	4.7	0.3	0.3	**28.4**
1979	5.9	7.1	12.0	35.3	**60.2**	20.8	4.9	0.3	0.4	**26.4**
1980	6.0	7.8	10.5	29.9	**54.2**	23.9	8.2	1.0	0.2	**33.3**
1981	4.5	6.2	16.5	32.0	**59.1**	21.1	8.8	0.3	0.3	**30.6**
1982	5.6	6.4	13.3	30.0	**55.3**	27.2	5.6	0.1	0.0	**33.0**
1983	3.4	7.0	14.2	38.6	**63.3**	20.2	6.1	0.3	0.0	**26.6**
1984	4.1	3.9	12.3	33.4	**53.6**	27.2	6.8	0.9	0.2	**35.1**
1985	4.7	4.7	12.2	32.0	**53.6**	23.4	8.7	1.6	0.2	**33.9**
1986	4.0	5.4	13.5	36.2	**59.0**	19.0	9.1	0.6	0.0	**28.7**
1987	5.7	5.8	13.7	31.4	**56.6**	19.6	8.8	0.6	0.2	**29.2**
1988	5.0	5.5	13.7	32.4	**56.7**	24.9	6.6	1.1	0.1	**32.7**
1989	4.8	6.2	13.1	24.8	**48.9**	25.2	9.9	0.6	0.0	**35.8**
1990	3.1	5.8	11.0	35.4	**55.3**	27.0	8.8	0.3	0.1	**36.1**
1991	5.2	6.3	13.8	28.5	**53.9**	28.8	5.6	0.8	0.3	**35.4**
1992	4.4	6.6	14.2	28.4	**53.6**	26.7	7.9	0.7	0.4	**35.6**
1993	3.2	4.7	15.0	28.7	**51.7**	25.9	9.4	0.8	0.1	**36.1**
1994	3.7	8.2	17.8	27.0	**56.7**	25.5	7.2	0.3	0.0	**32.9**
1995	4.9	8.5	12.9	30.5	**56.8**	22.6	8.1	0.5	0.1	**31.4**
			AVERAGE:		**56.4**			**AVERAGE:**		**32.0**

Bathymetry and Shoreline

Two sets of bathymetry data are used in this study. The first is obtained from the US Army Corps of Engineers (1995-2000) which conducted near-shore and offshore bathymetry measurement on the Galveston coast. Due to its sparseness, where on average the distance between two closest points is about 800 meters, only the offshore part of the data is used. For near-shore location, dense bathymetry data is required to capture the cross-shore variability of the sea bottom so that a realistic wave transformation may be carried out.

To fill the gap in the near-shore area, the second bathymetry data set is used. Texas A&M University at Galveston conducted cross-sectional measurement of near-shore bathymetry along the Galveston coast, starting from the South Jetty in the east to San Luis Pass in the west. The cross-sectional line starts from the water line to 2 kilometers offshore. With the exception of the first 2.5-kilometer beach stretch of the East Beach which has 200-meter spacing between two consecutive lines, all other lines space 800 meters apart. Although the bathymetry seems quite sparse alongshore-wise, it is very dense cross-shore-wise (two closest points in a line space less than 50 meters). The sparseness of the data in the alongshore direction cannot be problematic since the Galveston beach bottom profile is quite uniform alongshore-wise.

The shoreline data are obtained from the Bureau Economic of Geology which has made shoreline measurement and prediction for several years. For this study, the 1990 and 2001 shoreline are used. Both shorelines are nearly straight from east to west with azimuth about 34°. In both years the shorelines at the west end of the seawall eroded beyond the seawall, where the 2001 shoreline retreats 60 meters further landward than the 1990 shoreline. As one proceeds westward from the end of the seawall, however, both shorelines converge and nearly coincide which indicate that there was little shoreline change took place during that period.

Sediment Budget Analysis

Three assumptions are made in calculating the sediment budget analysis in this study. The first assumption is no alongshore transport occurs at the South Jetty; the second assumption is no cross-shore transport occurs along the Galveston coast; and the last assumption is the beach is under the equilibrium profile.

The first assumption may be problematic since the sand may pass over the jetty due to wave action and splashing. The second assumption may be valid over the entire island with the exception probably on the East Beach compartment where there is possible cross-shore transport from offshore to the beach (SONU *et al*., 1979). This assumption is backed up by the fact that over the period of 1990 to 2001 there was no big hurricane that could permanently wash the sand away from the beach (SNEDDEN, 1987).

According to Figure 13 and based on the conservation of sand mass, the net sand volume change in the indicated compartment is equal to the net sand inflow/outflow in the compartment, i.e.

$$\Delta V = Qin - Qout + q \,, \tag{4-1}$$

where ΔV is the net sand volume change in the compartment, Qin is the alongshore sediment inflow discharge, Qout is the alongshore sediment outflow discharge, and q is the source term (cross-shore transport, beach nourishment, etc.). Considering the beach is under the equilibrium profile (third assumption), the net sand volume change between time t and t', i.e. the area between profile t and t' of Figure 13a, can be easily calculated as

$$\Delta V = B(Db+Dc)\,\Delta sl \,, \tag{4-2}$$

where B is the compartment width and Δsl is the shoreline change over the period t-t'. Therefore, given the shoreline change one can calculate the sediment budget in a compartment and in turn in the whole compartments provided that at least one alongshore transport at the compartment wall is given.

In calculating the sediment budget in this study, the study area, 45.6 km long, is divided into 228 compartments of uniform width B = 200 meters. The calculation starts from the easternmost compartment, where at the east wall of this compartment (i.e. the south jetty) Qin = 0. Therefore, (4-1) simplifies to Qout = $-\Delta V + q$. Since there is no source term in the compartment, Qout = $-\Delta V = -200\times5\times36.8 = -36{,}800$ m^3. Note that the average shoreline change in this compartment is 36.8 meters. For the second compartment, Qout = $-36{,}800$ m^3 acts as Qin, and in a similar way, Qout of this compartment is calculated. Similar calculation proceeds to the other 226 compartments. However, since the groin field was nourished with 574,000 m^3 of sand and followed by three small nourishments of 54,000 m^3 each, q cannot be zero for the compartments in the groin field. For simplicity, the sand volume of the four nourishments is uniformly distributed over the groin field which gives q = 23,156 m^3, the total sand added in each compartment in ten years. The final result of this sediment budget calculation is presented in Figure 14. Positive and negative value of the alongshore transport in the figure means westerly and easterly transport respectively.

It is apparent, based on the 1990-2001 sediment budget analysis, that the alongshore sediment transport during the 1990-2001 period is divided into two main directions: in the first 12.4-km reach the transport direction is easterly and in the second 33.1-km reach the transport direction is westerly. The alongshore transport on the West Beach is nearly uniform about 180,000 m^3/year. Since in 1990 the shoreline on the West

Beach is nearly straight, the alongshore transport, according to CERC formula (2-2), must be constant provided that the source term in (4-1) is zero. The fact that the alongshore transport is constant (180,000 m^3/year) validates the assumption of no cross-shore transport made in the sediment budget analysis (particularly on the West Beach).

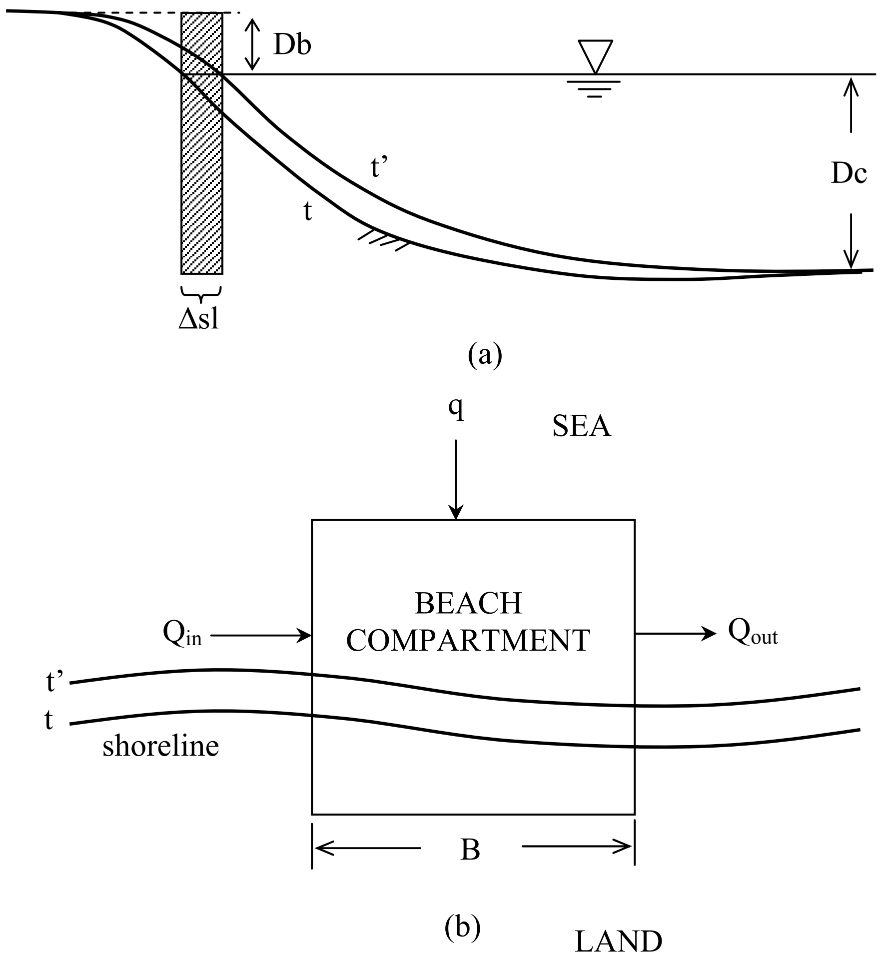

Figure 13. Sediment budgeting in a compartment.

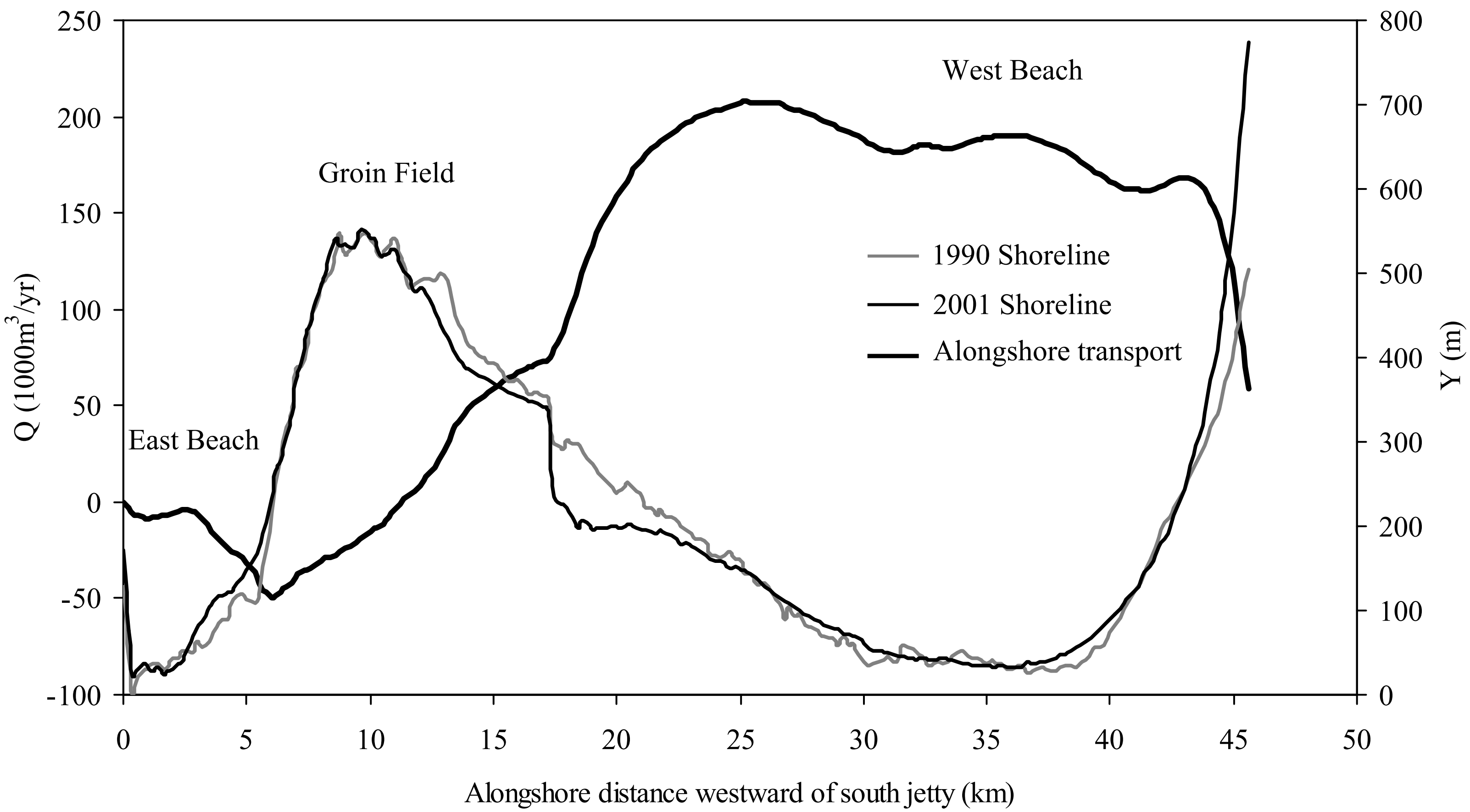

Figure 14. Annual alongshore sand transport derived from the 1990-2001 sediment budget analysis.

Potential Sediment Transport

The amount of sand that may potentially be transported by a certain wave along a coast is termed the potential sediment transport. Its calculation is based on the CERC formula and requires two major data: breaking wave condition and shore line data.

In this study the potential alongshore transport of the waves in Station-1079, which consists of the 1976-1995 hindcasted wave data, is calculated. In order to realistically determine this potential transport, the actual bathymetry rather than the equilibrium-profile based bathymetry is used in calculating the wave transformation from offshore to near-shore (see Wave Transformation Modeling). All structures are taken into account. Instead of using the 1990 and 2001 shoreline, the average of the two shorelines is taken as the shoreline position.

GENESIS with STWAVE as the external wave transformation model is exercised in the calculation of the potential alongshore transport. In order to maintain a relatively unaltered shoreline during GENESIS run, K_1 and K_2 are assigned to small value 0.01 which is about six to seven hundredth of the commonly used value 0.6-0.7. To compensate for the small K_1 and K_2 used in the calculation, the final transport calculation is multiplied by 70. The result of this calculation is depicted in Figure 15. Here, only the potential transport on the West Beach is considered since on the other part of the domain there are uncertainties such as the occurrence of cross-shore transport on the East Beach, the influence of the jetty to alongshore transport, and the influence of the groin field plus the seawall to the transport mechanism.

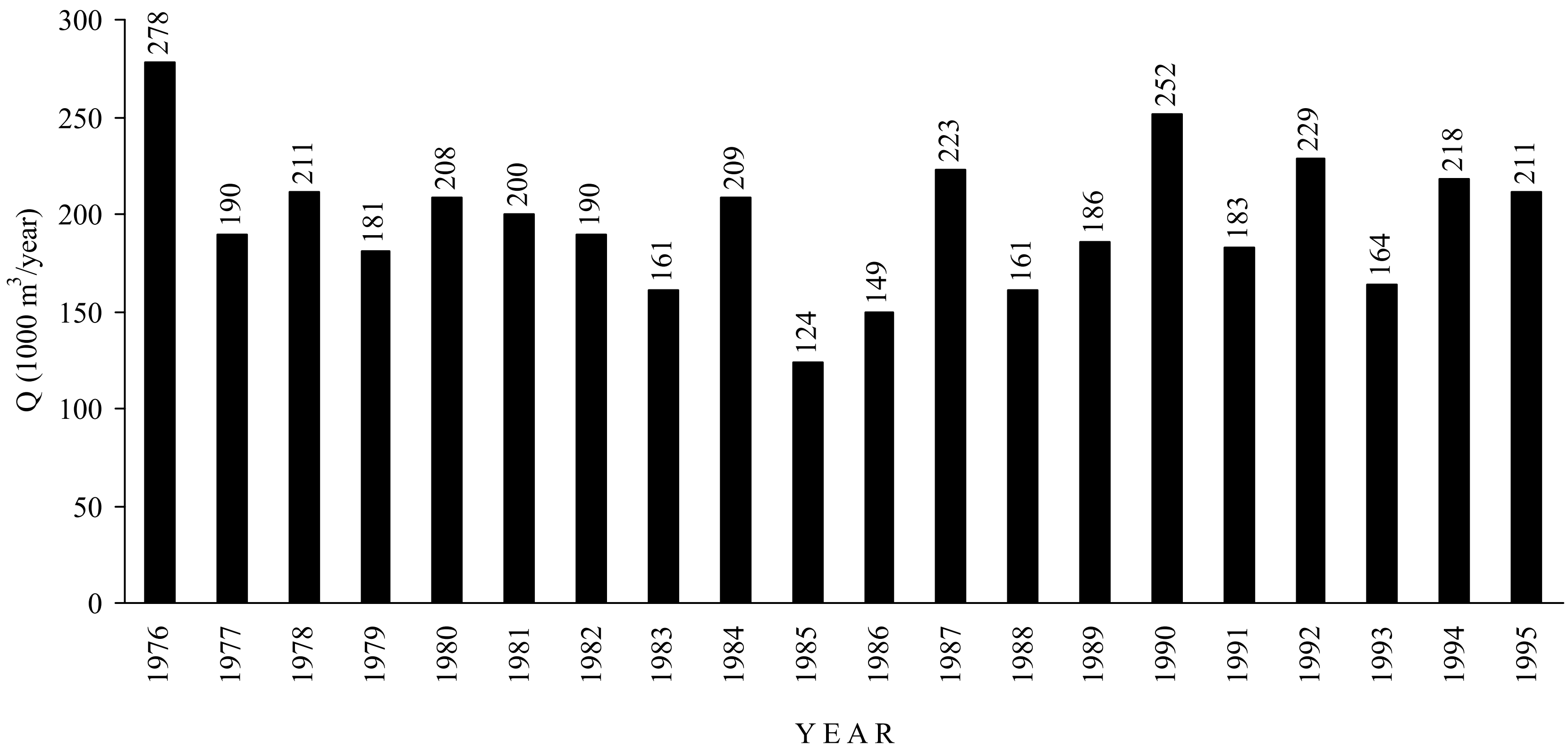

Figure 15. Potential alongshore transport along the West Beach, generated by the waves in Station-1079.

It may be seen that all potential alongshore transports are westerly, agreeing with that being indicated by the previous statistical and sediment budget analysis. On average the potential alongshore transport on the West Beach is 196,000 m^3/yr, close to the average annual sediment transport calculated in the sediment budget analysis (180,000 m^3/year). The first five closest potential transports to the sediment budget-inferred-transport are from the years of 1979, 1991, 1989, 1982, and 1977. The average of the four potential transports is 186,000 m^3/year, close to 180,000 m^3/year. Hence, the waves from those five years will be used for simulating the alongshore sand transport and the shoreline change in Galveston coast.

Wave Transformation Modeling

The wave data provided in the WIS Station-1079 is the wave condition at 15-m offshore station. The waves are transformed to near-shore reference line using STWAVE. In order to run STWAVE, a domain of calculation must be specified. In this study the domain of calculation is an 18,450-m wide by 43,500-m long rectangular area with its width and length running perpendicular and parallel respectively to the coast (see Figure 16). Note that this area covers the shoreline stretch that will be used for the alongshore sediment transport and shoreline change modeling. The average depth along the offshore side of the domain is 14.9 meters.

This domain is discretized into 370 shore-normal by 871 shore-parallel 50m×50m-squared grids. For simplicity and due to the limitation of the model capability, all the depths at grids under the shaded area (see Figure 16) are assigned to zero depth although some of the grids lie on the water. The depths at the rest of the domain are based on the bathymetry data as explained in the previous section. The two jetties on the calculation domain are modeled as land that sticks out to the sea. The depths at the tips of the two jetties are 8 meters.

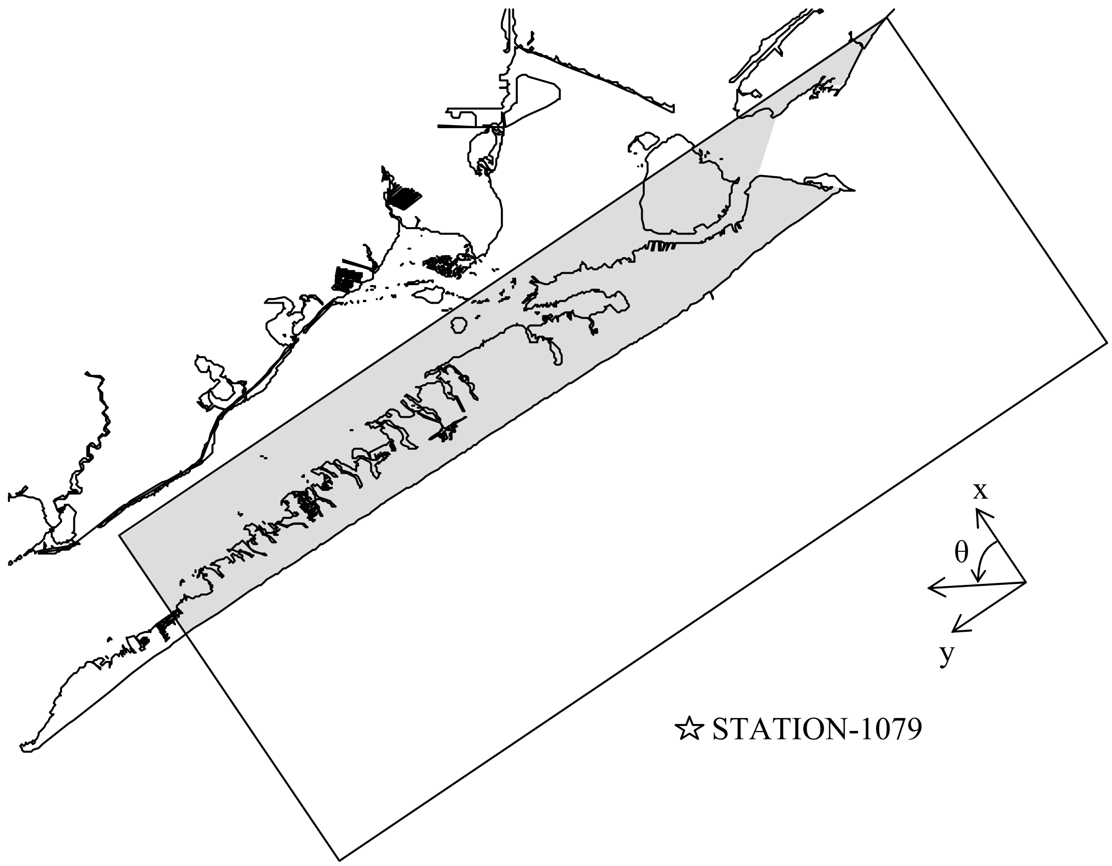

Figure 16. STWAVE domain for wave transformation modeling; depths on the shaded area are set to zero.

Prior to being used as input to the STWAVE model, the waves from Station-1079 (depth = 15 meters) are first transformed to the offshore boundary of the STWAVE domain (depth = 14.9 meters) using the WISPHS3 model (a simplified point-to-point steady-state, time-independent, spectral transformation), which is part of the wave modeling package in the CEDAS (Coastal Engineering and Design Analysis) program.

As explained in the GENESIS manual, the waves are to be grouped into several intervals of height, period, and direction. Instead of running STWAVE for the whole waves, which is time consuming, the mid-values of the heights, periods, and directions of the intervals are used as the wave input for STWAVE. For this purpose, the waves in Station-1079 are grouped into 11 wave height intervals, 13 wave period intervals, and 7 wave direction intervals (see Table 2). Note that the wave direction (θ) uses the angle convention as shown in Figure 14. The wave transformation calculation is carried out using the wave conditions on the "mid-value" column of the table.

Table 2. Wave intervals used in grouping the 1979, 1991, 1989, 1977, and 1982 waves.

H (meter)		T (second)		θ (º)	
Interval	Mid-Value	Interval	Mid-value	Interval	Mid-value
0.25 - 0.75	0.5	3 - 4	3.5	81 - 58.5	69.75
0.75 - 1.25	1.0	4 - 5	4.5	58.5 - 36.0	47.25
1.25 - 1.75	1.5	5 - 6	5.5	36.0 - 13.5	24.75
1.75 - 2.25	2.0	6 - 7	6.5	13.5 - –9.0	2.25
2.25 - 2.75	2.5	7 - 8	7.5	–9.0 - –31.5	–20.25
2.75 - 3.25	3.0	8 - 9	8.5	–31.5 - –54.0	–42.75
3.25 - 3.75	3.5	9 - 10	9.5	–54.0 - –76.5	–65.25
3.75 - 4.25	4.0	10 - 11	10.5		
4.25 - 4.75	4.5	11 - 12	11.5		
4.75 - 5.25	5.0	12 - 13	12.5		
5.25 - 5.75	5.5	13 - 14	13.5		
		14 - 15	14.5		
		15 - 16	15.5		

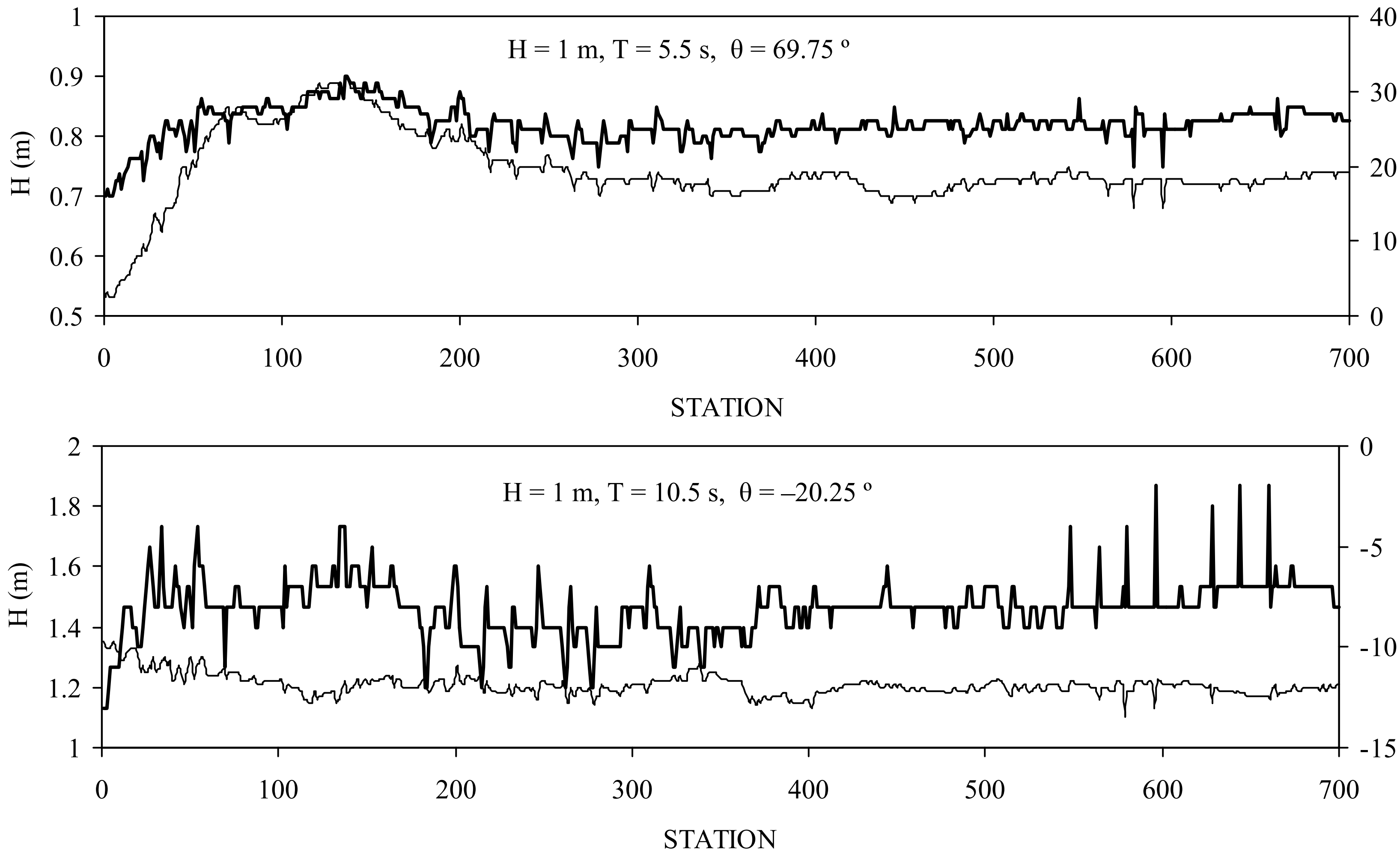

Figure 17. Typical wave height (light) and direction (heavy) in the near-shore stations.

Seven hundreds near-shore stations on average depth of 2.28 meters are installed parallel to the shore to store the wave transformation result. Since non-unit wave heights are used (see Table 2) in the transformation, the wave heights in the station must be divided by the corresponding wave heights in the table to get the wave transformation coefficients. These wave transformation coefficients will be used by GENESIS to determine the wave heights in the stations, i.e. by multiplying the wave heights with their corresponding transformation coefficients. Two typical plots of the STWAVE output (wave height and direction) along the stations are given in Figure 17.

Alongshore Transport and Shoreline Change Calculation

Domain Setup

The study domain covers 74% of the total Galveston coast, starting from the South Jetty in the east to 34,950 meters to the west. The domain encompasses all coastal structures present in the island, i.e. South Jetty, seawall, and groins and must be taken into account in the calculation.

Considering the inaccuracy of GENESIS for large spatial grid size (Chapter III), the study domain is discretized into 700 alongshore grids, where each grid measures 50 meters (equals the wave calculation grid size). In this configuration the south jetty is located at the first grid, the easternmost groin at 125th grid, the westernmost groin at 248th grid, and the other 13 groins in between. The east and west end of the seawall are located at the 95th grid and 346th grid respectively. This domain is depicted in Figure 18.

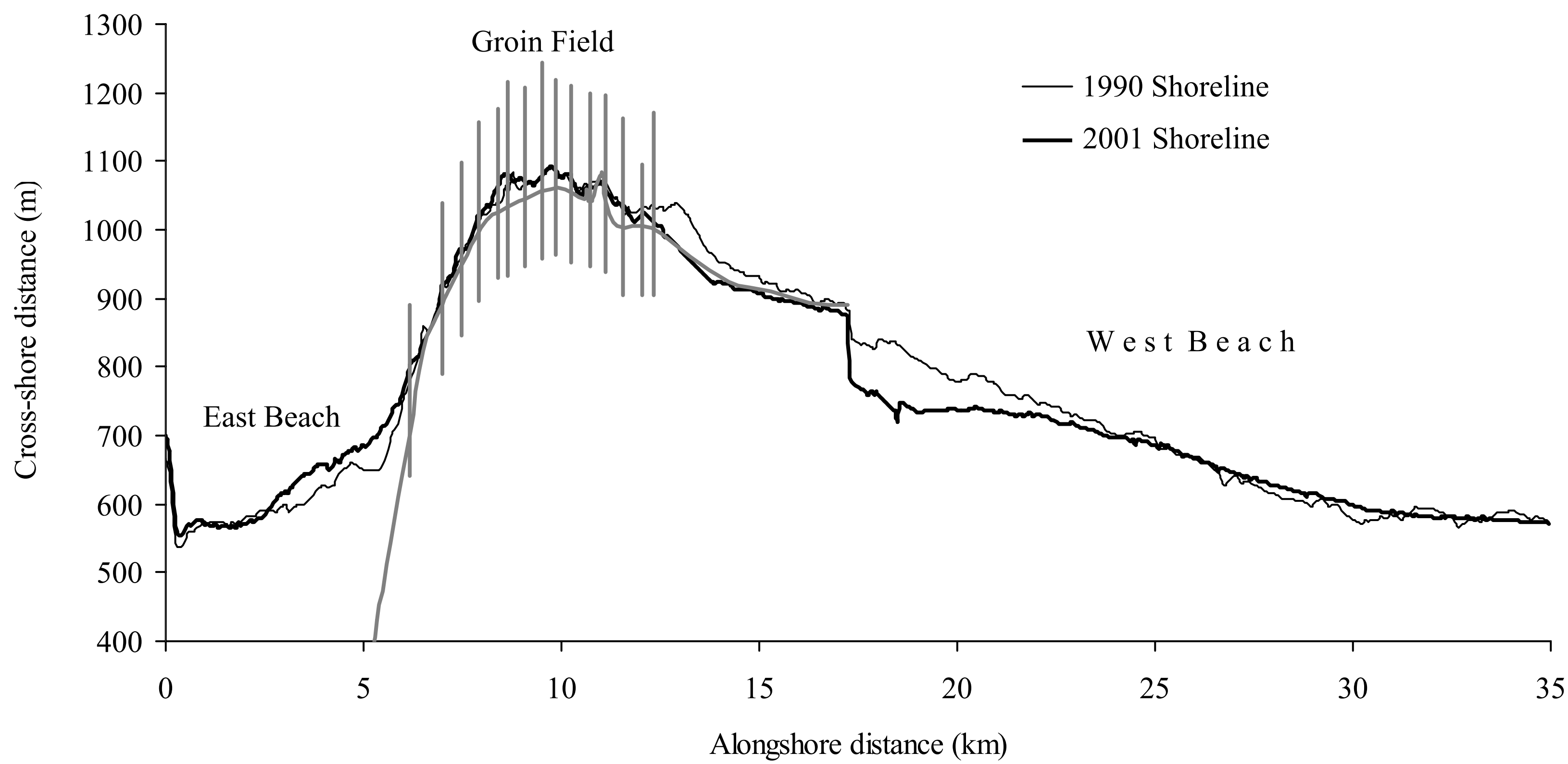

Figure 18. Domain setup for sediment transport and shoreline change calculation.

Boundary and Initial Condition

Two boundary conditions and one initial condition are required to run GENESIS. The first boundary condition is on the east side of the domain where the jetty is located. Therefore the gated boundary condition is suitably applied on this boundary. Since in the calculation of the wave transformation the effect of the jetty has been taken into account, i.e. by extending the land along the jetty alignment into the sea, the jetty/groin for this boundary condition is set as non-diffracting groin so that GENESIS does not carry out the diffraction calculation due to the presence of the groin on the boundary. The second boundary condition on the west side of the domain is set as pinned boundary condition because the 1990 and 2001 shoreline nearly coincide at this point, which means during that period there has been little shoreline movement. And for the initial condition, the 1990 shoreline is used.

Model Calibration

Here, GENESIS is calibrated against two calibration parameters, namely the sediment transport constant (K_1) which appears in the CERC formula and the south jetty permeability. All other parameters, grain size (D), depth of closure (D_C), berm height (D_B), and groin permeability p_{groin} are held constant.

The values of the constant parameters above are determined based on the observation and field measurement. Field observations along the Galveston beach show that the sand grain is very fine, much smaller than a millimeter. For this reason, D = 0.13 mm is chosen as the sand grain size. The quarry stones that comprise the groins in the groin field are so closely arranged that it is very unlikely that the sand transport can move through the groins; hence $p_{groin} = 0$. Two different bathymetry measurements at two different times indicate that the offshore bottom profile at the depth of around 3 meters is relatively stable, therefore the depth of closure $D_C = 3$ meters. Having found the depth of closure and considering the fact that the bottom (along Galveston beach) obeys the equilibrium profile, the berm height D_B is easily determined based on the two bottom profiles from the two measurements. Without going into further calculation, the berm height is found to be $D_B = 2$ meter.

Table 3. Calibration result.

K_1	K_2	p	E
0.60	0.6	0	18.1
0.65	0.6	0	17.7
0.65	0.6	0.05	14.5
0.70	0.6	0.06	13.8
0.75	0.6	0.06	13.4
0.77	0.6	0.06	13.3
0.78	0.6	0.06	13.2

The beach nourishment events in 1995, 1998, 1999, and 2000 are modeled here as beach fills, where for simplicity it is assumed that the sand is distributed uniformly in front of the seawall along the groin field, which spans about 6,150 meters. Therefore, the first nourishment is equivalent to 574,000/(5×6,150) = 18.5 m beach width increase and the other nourishments (54,000 cubic yards) are equivalent 54,000/(5×6,150) = 1.7-m beach width increase. Note that the 0.76 factor is the conversion factor from cubic yard to cubic meter. The result of this calibration is presented in Table 3. It can be seen that

$K_1 = 0.78$ and the groin permeability p = 0.06 result in the smallest error E = 13.2. Here, K_1 turns out to be similar to the SPM recommended value.

Error Determination

Since the domain used in this study is nearly similar to the idealized domain used in the first analytical solution of the Model Test, the error estimation in the calibration calculation may be assessed using the error analysis given in the Model Test. For that purpose, a wave of the form given in the first analytical solution must be determined so that the alongshore sand transport generated by this wave is similar (or close to) to the transport generated by the waves used in the calibration process, i.e. the 1979, 1991, 1989, 1977, and 1982 waves. This is done by minimizing the following objective function with respect to $\overline{H}$ and α_{a0}:

$$f_{obj} = \left| \sum_{i=1}^{N} H_{bi}^{2.5} \sin(2\theta_{bs}) - \sum_{i=1}^{N} \overline{H}^{2.5} \sin(2\theta_i) \right| , \qquad (4\text{-}3)$$

where

H_b = breaking wave heights of the waves in 1979, 1991, 1989, 1977, and 1982

θ_{bs} = breaking wave angle to the shoreline

$\overline{H}$ = wave height in the analytical solution

θ_i = wave angle in the analytical solution = $\alpha_{a0}(1+\sin(2\pi t/Tdir))$, Tdir = 5 years.

It is found that $\overline{H}$ = 0.29 m and α_{a0} = 15.7° minimize the objective function (4-3). With those $\overline{H}$ and α_{a0} values, Δx = 50m, t = 11¼ years (simulation length) , and K_1 = 0.78, the dimensionless parameter Π is calculated to be 0.0022 and from Figure 4 the error at 300 meters from the jetty is 14%. Therefore the shoreline calculation in this calibration may have an error about 14%. To compensate for this error, K_1 may be reduced by factor of 14%, which gives $K_1 = 0.67$.

Calculated Alongshore Sediment Transport and Shoreline Position

The calculated alongshore sediment transport and shoreline based on $K_1 = 0.78$ are given in Figure 19 and 20. The overall transport pattern based on the GENESIS calculation and sediment budget analysis show agreement. On the West Beach the GENESIS-calculated annual alongshore transport is 188,000 m^3/yr. It agrees very well to transport calculation based on the sediment budget analysis (180,000 m^3/yr). HALL (1976) found that the transport on the West Beach was 116,000 m^3/yr, 39%, lower than the calculation in this study. The difference between Hall's transport and this study's calculation is due to the fact that GENESIS includes the porosity of sand ($p = 0.4$) in its transport calculation (see (2-2) and (2-3)) while Hall's does not. If the similar porosity is included in Hall's calculation, the transport will be 189,000 m^3/yr, similar to GENESIS'. Non-zero permeability of the jetty in the calibration allows sand transport across the jetty. Since the net is westward (positive), it results in an upward-shift of the transport curve as compared with the same curve from the sediment budget analysis.

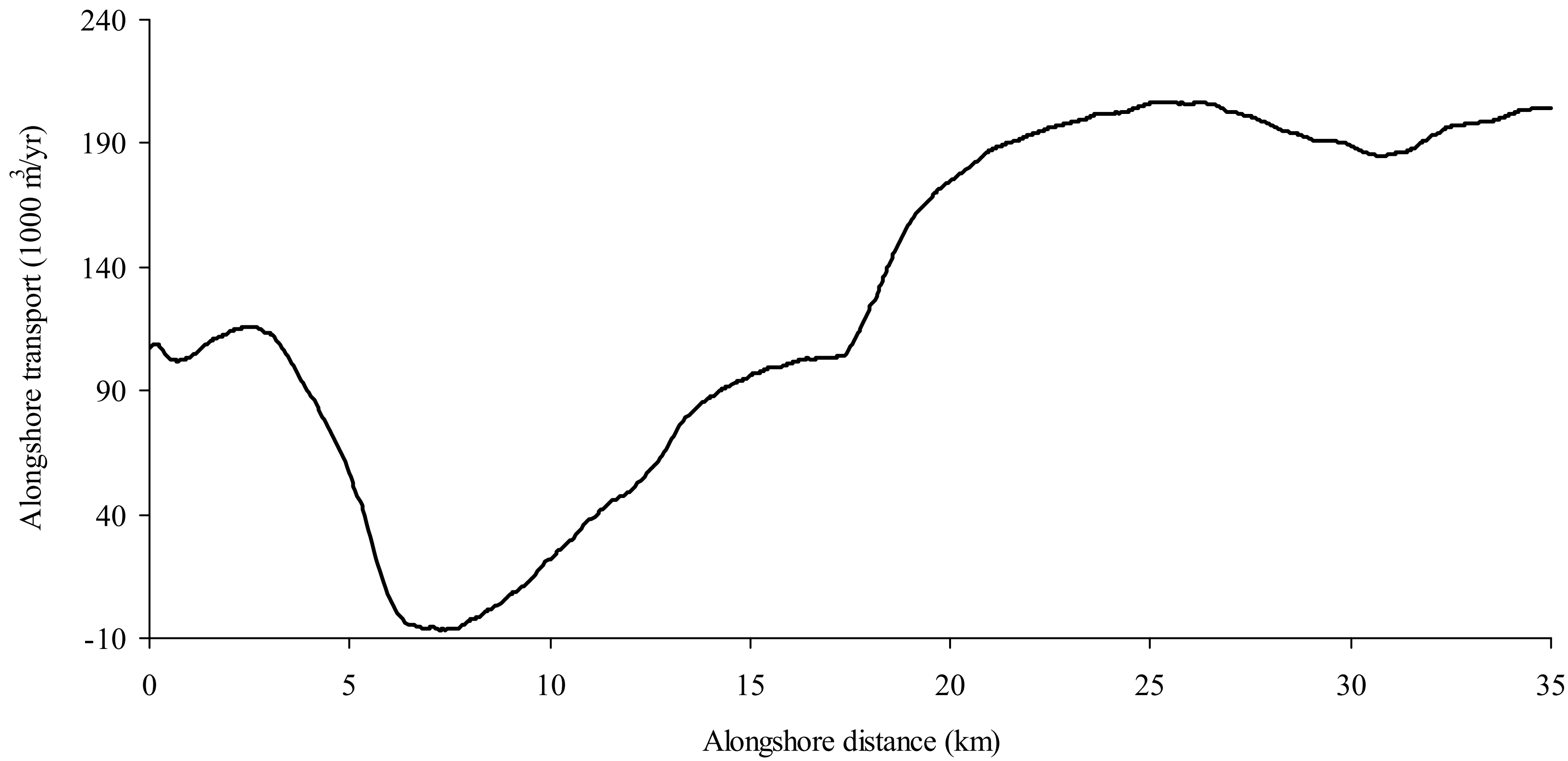

Figure 19. Annual net-alongshore transport calculated based on the best calibration parameters.

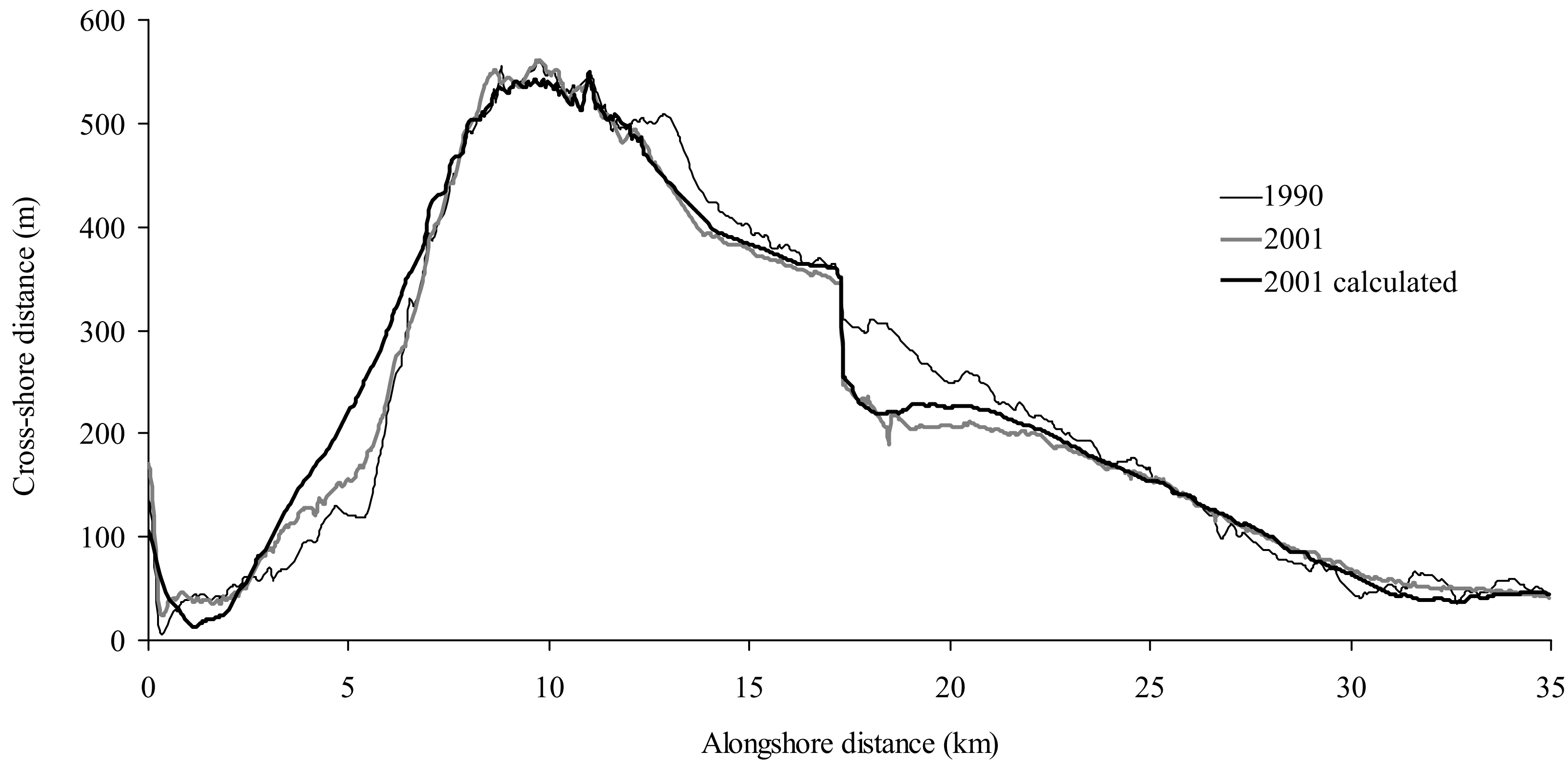

Figure 20. 2001 shoreline position calculated based on the best calibration parameter.

GENESIS adequately predicts the final shoreline position on the West Beach. At the end of the seawall, the amount of shoreline erosion calculated by GENESIS is nearly equal to the actual shoreline erosion. On the East Beach (particularly the 4km-reach, 3 km west of the South Jetty), however, GENESIS does not calculate the shoreline change satisfactorily. The disagreement may be due to two factors:

1. Lack of information of the correct amount of cross-shore transport which is suspected to occur on the East Beach.
2. Some of the waves are subject to diffraction due to the jetty, while the STWAVE model deals with diffraction in a simple fashion.

CHAPTER V

EVALUATION OF EROSION CONTROL'S PERFORMANCE ON GALVESTON COAST

The performance of two erosion controls on the Galveston beach in the period of 2001 to 2011 will be examined. The two erosion controls consist of:

1. Beach nourishment in the groin field.
2. Beach nourishment in the seawall end proximity.

Although the evaluation is conducted for the future period, the same wave data set as in the calibration process (from the years of 1979, 1991, 1989, 1977, and 1982) is utilized. The validity of using this wave data set to calculate the shoreline change for the future time period may be questionable and needs to be backed up by a rigorous wave forecasting method. However, in this study, for simplicity's sake, the wave climate in the period of 1976-1995 is assumed to persist throughout the next ten or twenty years. This assumption is supported by the fact that the wave climate in each year of the 1976-1995 period, as may be seen in Figures 8 to 12, is similar to each other and is likely to persist throughout the next ten or twenty years.

In all these evaluations, the beach nourishment in the groin field in the winter 2002 is included. This nourishment causes the shoreline to advance by 1.73 meters.

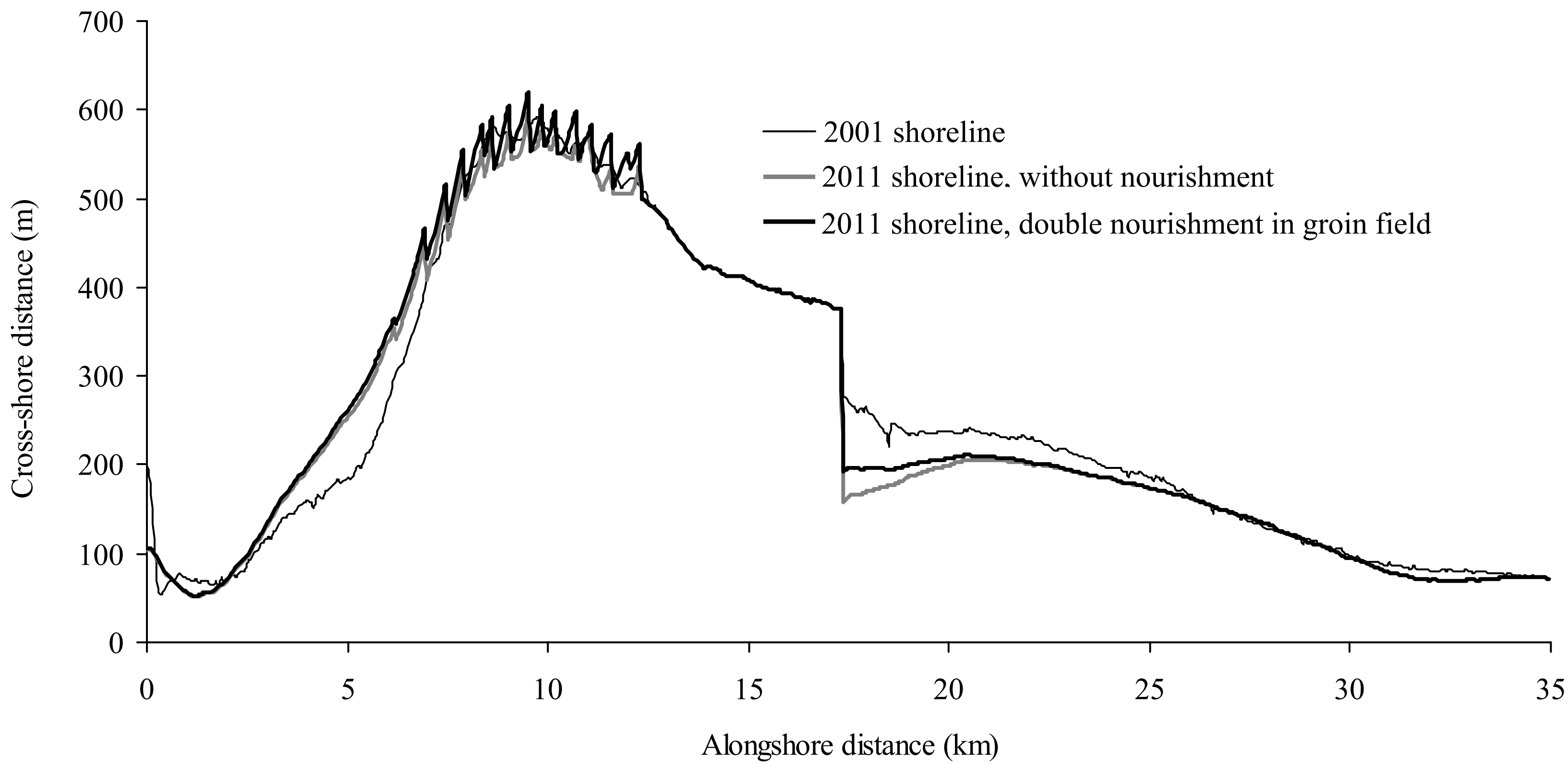

Figure 21. 2011 shoreline position calculated based on the groin field nourishment scenario.

Beach Nourishment in the Groin Field

Here, 100,000 m^3 of sand per year is used to nourish the beach in front of the groin field. It is twice as much as the nourishment volume used in the nourishment events in 1998, 1999, and 2000, where 57,000 m^3 of sand per nourishment event was spread along the groin field. Each nourishment event of this scenario produces a 3.3 meters shoreline advance and modeled in GENESIS as an annual beach fill.

The calculated shoreline position (in 2011) based on this erosion control scenario is presented in Figure 21. On average the shoreline accretion along the groin field is 16.3 meters in ten years. The impact of the nourishment on the hot spot area in the vicinity of the seawall end in the west is to reduce the erosion rate by 25%. This is due to the fact that some fraction of the nourishment sand is transported from the groin field to the west (particularly to the hot spot). On the contrary, the West Beach and East Beach are not affected by this nourishment. This fact can be clearly seen in Figure 21, where the 2001 and 2011 shoreline coincide along the West Beach and East Beach.

Beach Nourishment in the Seawall End Proximity

In this erosion control scenario, annual beach nourishment is applied on the hot spot area and the proximity in the period of 2001 to 2011. The nourishment area extends 1,500 meters to the west from the end of the seawall. Sensitivity analysis on the shoreline change with respect to the sand volume used in the nourishment is conducted. For this purpose, seven different nourishment volumes are used. These volumes are shown in Table 4. The shoreline advance produced by the nourishment is also included in the table.

Table 4. Beach nourishment volume (m^3) and shoreline advance (m) used in the beach nourishment scenario on the hot spot area.

No	Volume	S.L. advance
1	40,000	5.33
2	50,000	6.67
3	60,000	8.00
4	70,000	9.33
5	80,000	10.67
6	90,000	12.00
7	100,000	13.33

The simulation result of this scenario is shown in Figure 22. This figure shows that only the hot spot area is impacted by the nourishment. The West Beach, East Beach, and the groin field are not affected. As expected, the more sand dumped to the hot spot area, the less erosion happens. The erosion reduction associated with the nourishment on the hot spot proximity is shown in Figure 23. The erosion reduction factor linearly varies with the amount of beach nourishment volume.

In making the erosion control simulation above, it is assumed that during the simulation period (2001-2011), no big storm/hurricane occurs. Storm/hurricane can permanently wash away the sand from the beach (SNEDDEN, 1987) and produce an episodic shoreline change. If such a storm/hurricane occurs, the same amount of sand as washed away by the storm/hurricane is required to compensate for the erosion.

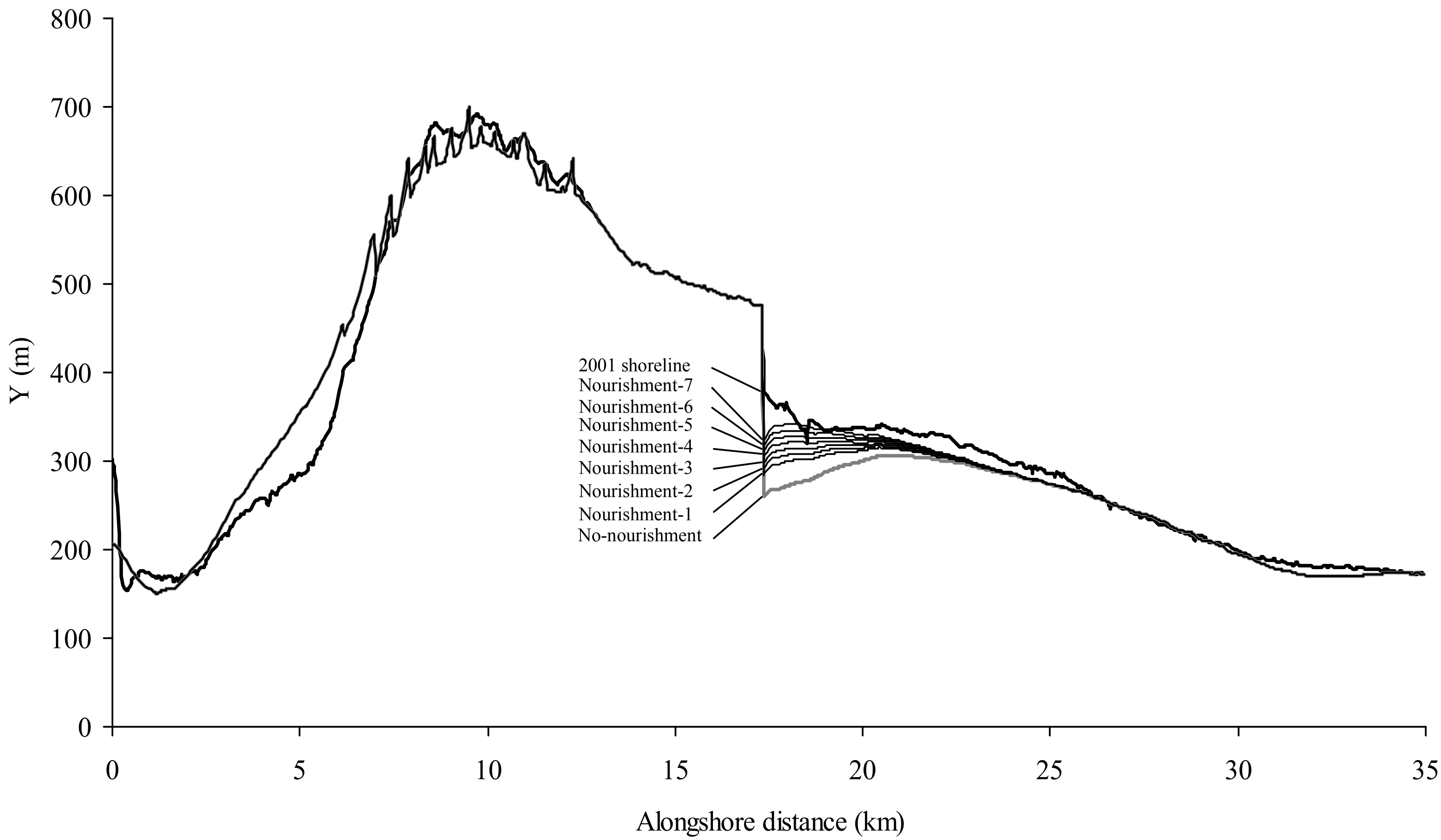

Figure 22. 2011-calculated shoreline position based on the scenario of the beach nourishment on the hot spot area.

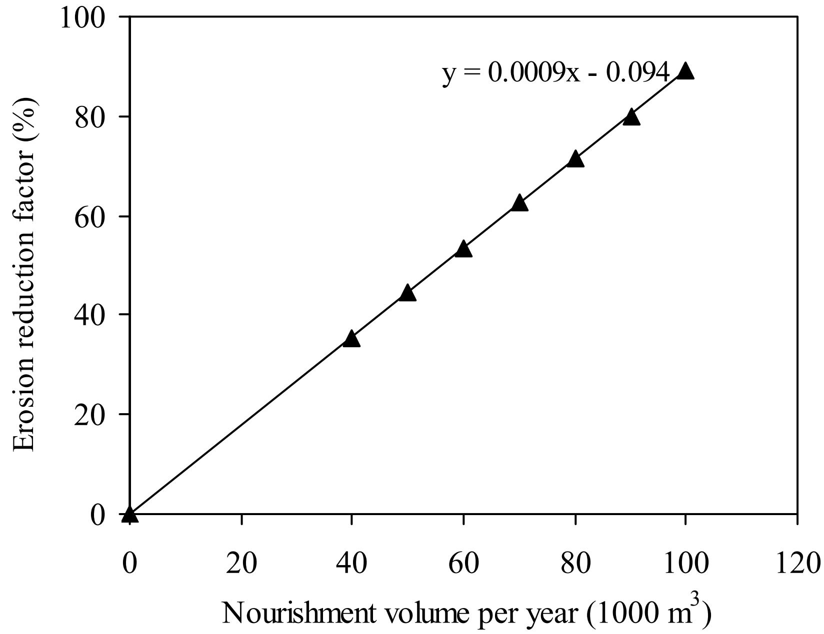

Figure 23. Erosion reduction factor associated with the beach nourishment in the hot spot area.

Other erosion control practices such as non-diffracting groin/T-groin on the West Beach and offshore-breakwater offshore of the West Beach are not discussed here because such coastal structures simply shift the erosion problem further west. For instance, Figure 24 demonstrates the erosion shifting further west due to the construction of offshore breakwaters offshore of the hot spot area.

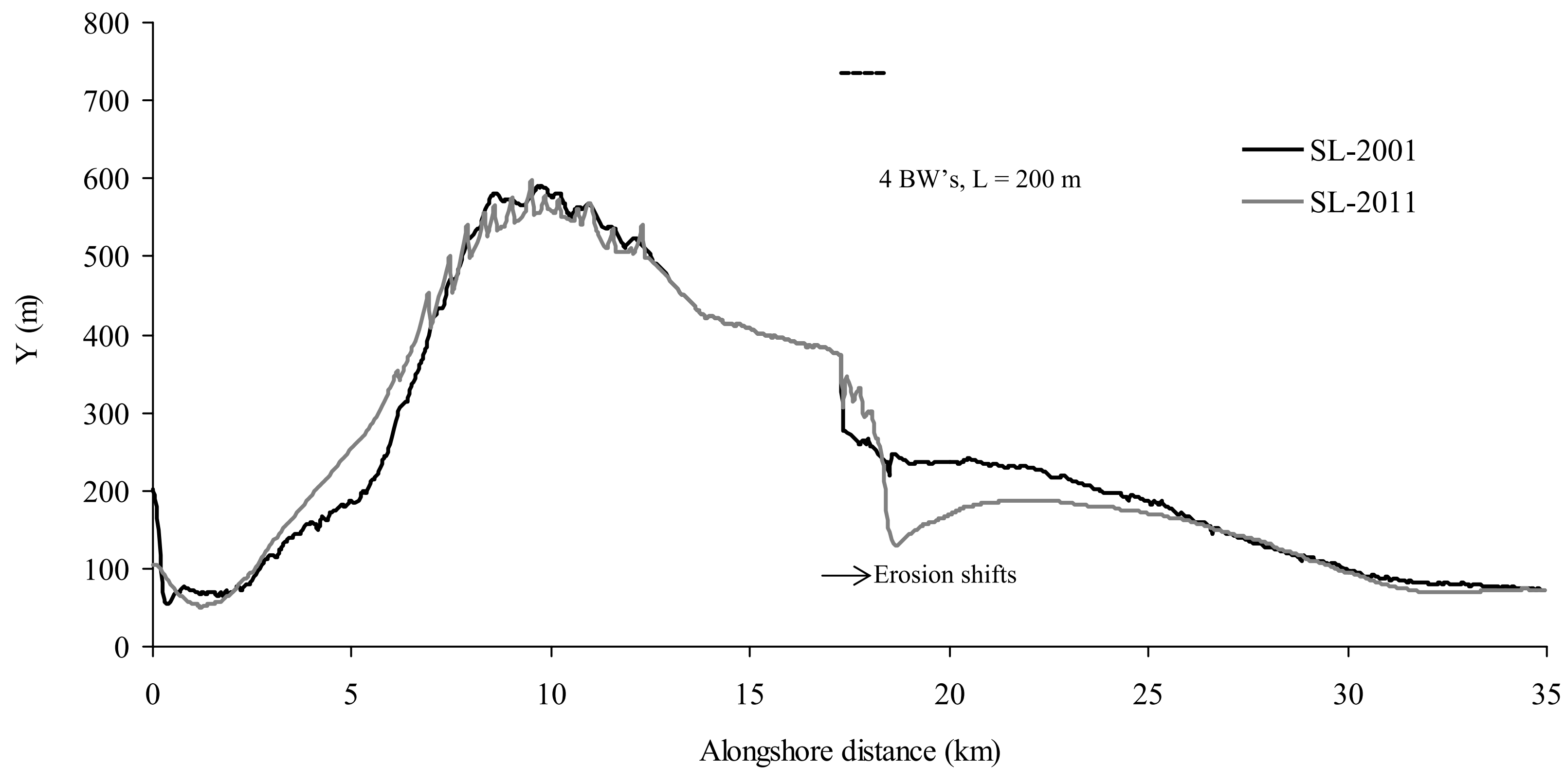

Figure 24. Breakwater construction to protect the hot spot area shifts the erosion problem further west.

CHAPTER VI

CONCLUSION

The study of the alongshore sediment transport and shoreline change analysis on the Galveston beach shows good result, especially on the West Beach and Groin Field. The key success to modeling the alongshore sediment transport and shoreline change lies in:

1. The ability to recognize the assumptions made in the GENESIS model and the ability to find out whether or not the assumptions are valid in the domain of study.
2. The ability to determine the proper wave data set that can generate the desired alongshore transport.

One crucial assumption which is valid along the Galveston coast during the period of study (1990-2001) is the assumption of no cross-shore transport made in GENESIS. This is supported by the fact that during this period, no big storm/hurricane occurred. Big storm/hurricane can generate cross-shore transport.

The sediment budget and potential alongshore transport are used in this study to determine the proper wave data set for modeling the alongshore sediment transport and shoreline change. This is done by selecting the wave data set whose potential alongshore transports resemble the sediment-budget-inferred transport. This method provides a systematic way of selecting the wave data set for use in the analysis of shoreline evolution and proves to work adequately in this study. The application of this technique on the wave data base in the WIS-Station-1079 tells that the waves from the years of 1979, 1991, 1989, 1982, and 1977 are the most representative wave data set for studying the alongshore sediment transport and shoreline change in the period of 1990 to 2001. Those five years wave data set produces as much as 194,000 m^3/year of alongshore sand transport on the West Beach. This is very similar to the alongshore transport produced in the sediment budget analysis (180,000 m^3/year).

Rigorous wave transformation model that accounts for diffraction is recommended to use whenever any diffracting structure (such as the South Jetty) presents in the domain of study. Otherwise, agreement between calculated and measure shoreline is hard to achieve. For instance, in this study agreement between the calculated and measured 2001-shoreline in the vicinity of the South Jetty is poor. One possible reason is that STWAVE (which is used as the external wave transformation model) does not correctly calculate the diffraction due to the jetty.

In the calibration process, K_1 = 0.78 turns out to be the best calibration parameter. The Shore Protection Manual (1980) suggested the same value to be used in the CERC transport formula while HALL (1976) used K_1 = 0.81 in his transport calculation.

Another important issue is the numerical accuracy of GENESIS when it is used for the alongshore sediment transport and shoreline change analysis. The error, which is 14% in this study, can be compensated by adjusting the transport constant K1.

The performance of any erosion control to resolve the serious erosion problem along the Galveston beach should be first assessed before applying it on the site. Among several possible erosion controls (beach nourishment, offshore breakwater, groins, etc.), beach nourishment proves to be the best alternative to resolve the erosion problem (particularly on the hot spot area and West Beach). Here the effectiveness of the beach nourishment on the Groin Field and the hot spot area was tested. It was found that by adding twice as much as the volume of sand used in the beach nourishment event of the 1998, 1999, and 2000, to the groin field, this area gains about 1.6 meters of shoreline advance per year. Moreover the erosion rate in the hot spot area is reduced. The nourishment on the hot spot proximity reduces the erosion of the shoreline in this area; the erosion reduction factor varies linearly with the volume of nourishment. With 40,000 m^3/year of sand spread over a 1.5 km stretch of the beach on the hot spot area, the erosion can be reduced by 35% and it increases to 89% if 100,000 m^3/year is used.

The construction of groins and offshore breakwaters on the West Beach do not resolve the erosion problem, instead shifts it further west.

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